Hawes —

Corrugated Bars for Reinforced Concrete 1905

15-00

D. E. GARRISON, President. D. E. GARRISON, Jr., Sec'y and Treas. A. L. JOHNSON, M. Am. Soc. C. E., Chief Engineer.

St. Louis Expanded Metal Fireproofing Co.

Suite 606 Century Building,

ST. LOUIS, MO.

SOLE AGENTS FOR THE SALE OF

CORRUGATED BARS.

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ST. LOUIS EXPANDED METAL FIRE PROOFING CO.

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INTRODUCTION

The year 1904, just closed, has seen many advances in the field of reinforced concrete construction in general, and in corrugated bars in particular, which the following pages will serve to indicate.

We have patented a new type of corrugated bar, one having a constant cross section, shown on page 13, but will continue to manufacture the old style bar, as, although this bar has some material not available for strength, there are occasions when it will better serve the purpose than the latter type. Both bars are now manufactured in soft, medium or high carbon steel.

In the general field of reinforced concrete the predictions which we have been making for several years are now being verified. These relate to the advantage of a high elastic limit, and the advisability of a mechanical bond.

ELASTIC LIMIT.—There has been a great deal of discussion as to the reliability of Considère's conclusions as to the ability of concrete to stretch without rupture ten or fifteen times as much when reinforced with metal as when unreinforced, and there is certainly reason to suspect that his results are incorrect, at least not true in all cases. Concretes will vary in stretchability, depending upon the materials used, and upon their wet or dry condition. Dry concrete is brash, much like dry timber, and laboratory results on bone-dry specimens would not be representative of open-air structures. Certainly the metal acts as an integrator, enabling us to obtain at all sections the maximum stretch of which each section is capable, instead of, at all sections, only that of which the weakest section is capable. This will give a proportionate elongation, according to the latest investigations, of from .0004 to .0005, equivalent to a stress in the metal of from 12,000 to 15,000 pounds per square inch.

All of this analysis, however, is really beside the mark. A stress in the imbedded metal of 50,000 pounds, if inside the elastic limit, can result in no harm to structures reinforced with such a material as the corrugated bar. In such a case, even if the cracks were as far apart as six inches, they would only have a width of .01″ and, at a depth of two or three inches below the surface, even if this crack extended clear down to the bar, as it might do if plain bars were used, it is doubtful if the mild acidity of the carbonic acid in the air could cor-

rode the metal between two such strongly alkaline surfaces only .01″ apart. However this may be with the plain bar, it is certain that the crack could not extend down to the surface of a corrugated bar, as this would involve a slip along the bar, which would necessitate the shearing off of the concrete entering the recesses on the bar's surface, a condition only to be obtained with the demolition of the structure.

The true function of the metal therefore is not to *prevent* cracks, but to subdivide a given stretch into a great many cracks. If this is done, and a corrugated bar used, it is of no consequence when the cracking first begins, nor what the stress in the metal reinforcement is, so long as it is inside the elastic limit, be that limit however high.

Inside the elastic limit, then, we have no damage. Beyond this limit, however, we encounter cracks of very large extent, which would soon result in the collapse of the structure. Therefore, in our judgment, the factor of safety for reinforced concrete should be based upon the capacity at the elastic limit of the metal reinforcement, and should be, generally speaking, not less than four.

The building laws of many cities which now allow a working stress in the metal reinforcement of 16,000 pounds per square inch, whatever kind of metal it may be, even though it has an elastic limit of not over 30,000 pounds per square inch, are examples of reckless disregard of the public safety.

If, therefore, we are safe inside any reasonable elastic limit, and our working stress is this limit divided by our factor of safety, which should be not less than four, then it is wise and economical to have as high an elastic limit as possible consistent with such ductility as may be required by the work in hand. Generally little ductility is needed, but in some cases where much cold bending has to be done, medium, or even soft steel might be required, and all three grades we are prepared to furnish.

MECHANICAL BOND.—There are three influences affecting the adhesion of cement to a metal surface, as follows:

1°. Breuillié, at La Châinette, reported some investigations in Annals des Ponts et Chaussées for 1900, which showed that soaking in water for nine months reduced the adhesion of concrete to metal from one-half to two-thirds.

2°. Prof. Schule, who now occupies the position at Zurich formerly held by Prof. Bauschinger, reported at the International Engineering Congress at St. Louis in October, 1904, that when the reinforcing bars were stressed, even though inside the elastic limit, the cross section was slightly reduced. Inasmuch as the adhesion consists, simply, in the entering by the cement particles into microscopical pores on the surface of the metal, any shrinkage of the cross section of the metal, however slight, was sufficient to materially affect the value of this adhesion.

3°. In our experience we have had cases of rupture of the adhesion with plain bars after eight years' use, where the structure was not wet, nor did the stress in the bars ordinarily amount to much, this failure being due entirely to vibrations and shocks.

In open-air structures all three of these influences will generally be found working at the same time. Starting with 500 pounds per square inch adhesion, suppose only one-half this is lost by being wet much of the time, this leaves 250 pounds. If one-half of this is lost by shrinkage of the cross section of the metal, due to stress in same, we then have only 125 pounds. Taking a factor of safety of four, and making no allowance whatever for vibrations and shocks, which alone are sometimes sufficient to destroy the whole of the adhesion, we have an allowable working stress for adhesion of 30 pounds per square inch. For a rod of 1″ diameter this means about 1200 pounds per lineal foot, which, to develop a working stress in the metal of 12,000 pounds per square inch, would require an anchorage of ten feet in which no other increment could be added! Such a requirement in practice would be absurd and impossible, generally speaking.

That foreign engineers, who have been mainly responsible for the use of plain bars for concrete reinforcement, are coming to realize the unreliability of adhesion alone, is indicated in many ways, chief of which is that the specifica-

tions prepared about a year ago, covering all this kind of work in the German Empire, state that "the bond shall, so far as possible, be of a mechanical nature." Up to that time there had been practically nothing used but plain bars. Further, it is noticeable that most of the French companies are now turning up their rods at the end or using some similar device, though what advantage is to be gained by turning up a three-quarter inch rod sixteen feet long an inch or two at the end, it is hard to realize.

Foreign engineers, as a matter of fact, have not had the experience that we have. Their beam work, in which alone these weaknesses develop, dates back only eight or nine years, while in the United States we have been building beams almost continuously since 1875. As it has taken eight years for this weakness to develop in some of our own work, and as abroad they first used mortar instead of concrete, which gives a stronger adhesion, it may be said that the time is only just arriving when we might expect them to discover the necessity of using other means of obtaining a reliable bond. And as before stated, these expectations are now realized.

When the unreliability of the adhesion is admitted, then it becomes necessary to have a mechanical bond that will avoid all splitting tendency on the concrete. This requires, with mathematical certainty, that the side of the ribs on the bar shall not vary from a plane at right angles to the axis of same by

an amount greater than the angle of friction between the concrete and metal, which is, generally speaking, about 45°. The corrugated bar is the only one in the market that fulfills, or that can ever fulfill, this condition, as our patent covers all bars that can be rolled in which the condition is complied with.

Summing up the situation, the corrugated bar has the following vital points of advantage over plain bars, and over all other types of bar reinforcement:

1°. Its elastic limit being high (unless by special requirement) enables a higher working stress to be used than should be used for soft steel bars, taking, therefore, proportionately less metal.

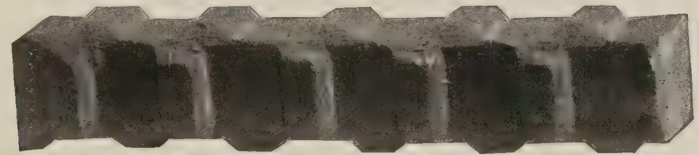
2°. Cracks in the concrete can not penetrate to the corrugated bar so long as the stress in the steel is inside the elastic limit.

3°. Soaking in water concrete reinforced with corrugated bars does not injure their bond.

4°. Reduction of the cross section of these bars, due to tension stress inside the elastic limit, in no way reduces their effective grip on the concrete.

5°. Vibrations and shocks do not impair their bonding value.

6°. Being formed by rolls while hot, the bars are all alike, the shape of each piece not depending upon the personal equation of some workman.



THE CORRUGATED STEEL BAR

WAS AWARDED THE

GOLD MEDAL

BY THE SUPERIOR JURY

LOUISIANA PURCHASE EXPOSITION

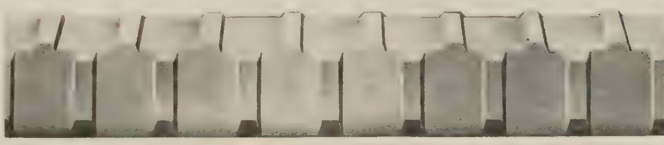
ST. LOUIS EXPANDED METAL FIREPROOFING CO.

CENTURY BUILDING GENERAL AGENTS ST. LOUIS, U. S. A.

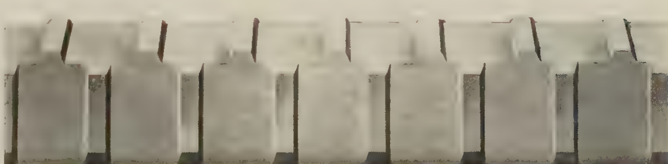
½″□ Bar. Net Section 0.18□″; Weight 0.64 lbs. per ft.

¾″□ Bar. Net Section 0.37□″; Weight 1.35 lbs. per ft.

⅞″□ Bar. Net Section 0.55□″; Weight 1.95 lbs. per ft.



1″□ Bar. Net Section 0.70□″; Weight 2.70 lbs. per ft.



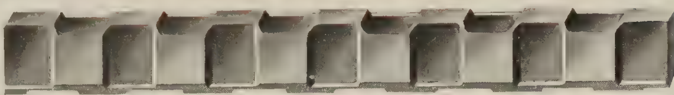
1¼″□ Bar. Net Section 1.07□″; Weight 4.00 lbs. per ft.

Old Style Corrugated Bars.

A variation in weight of 5% either way is required.

1/4" Bar. Net Section .06 □", Weight 0.21 lbs. per ft.

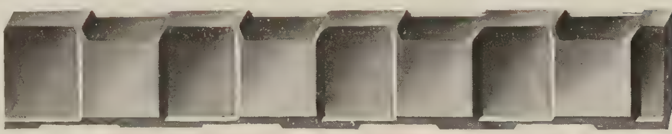
1/2" Bar. Net Section .25 □", Weight 0.85 lbs. per ft.



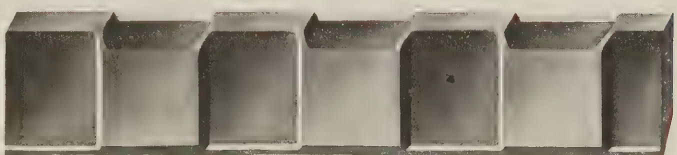
5/8" Bar. Net Section .39 □", Weight 1.33 lbs. per ft.

3/4" Bar. Net Section .56 □", Weight 1.91 lbs. per ft.

7/8" Bar. Net Section .77 □"; Weight 2.60 lbs. per ft.

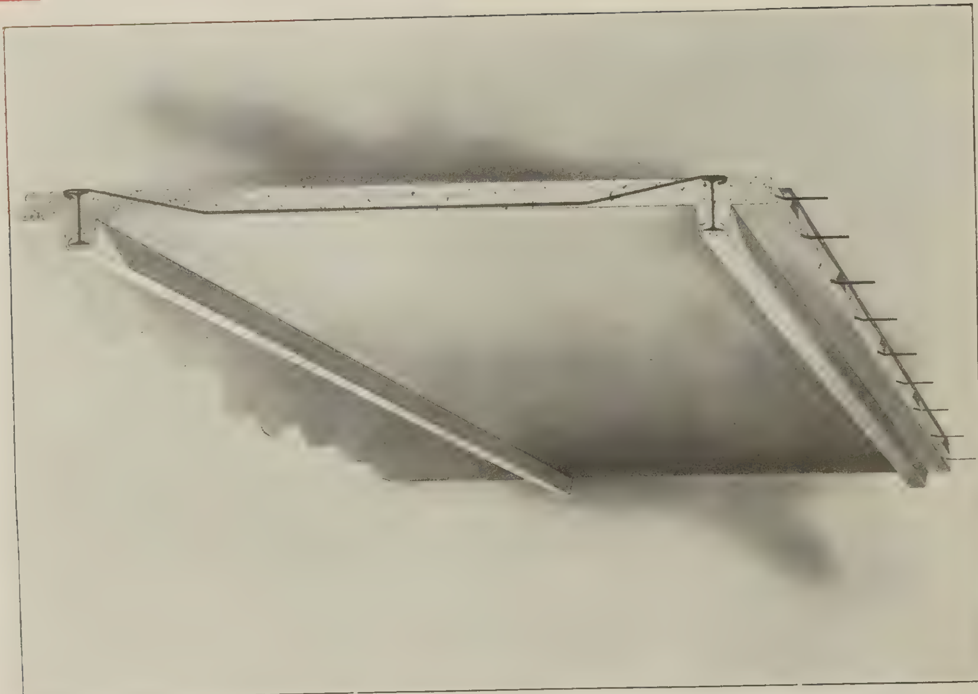


1" Bar. Net Section 1.00 □", Weight 3.40 lbs. per ft.

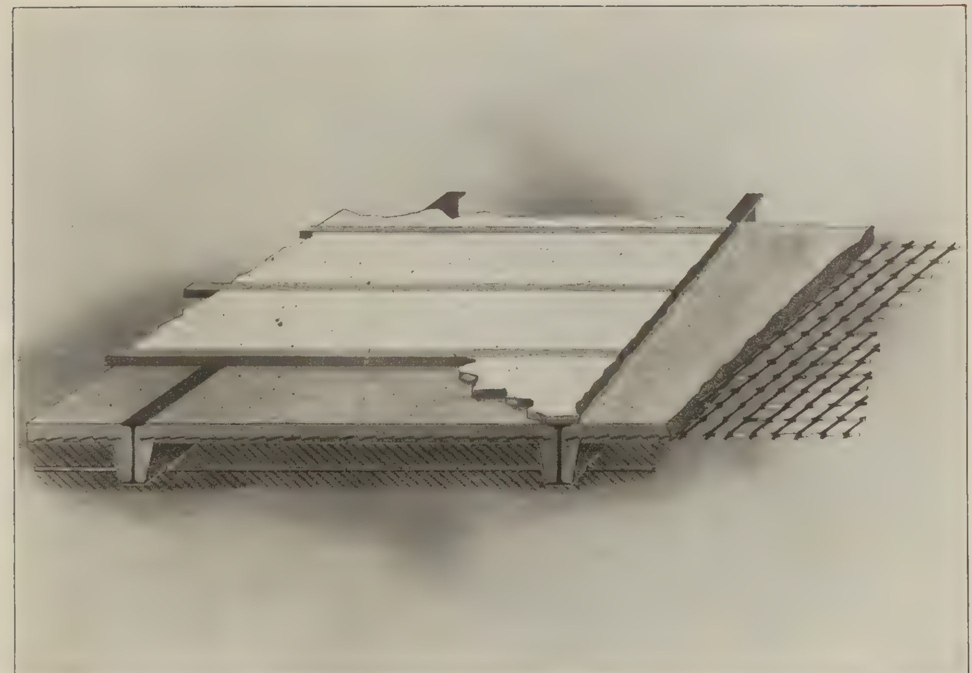


1 1/4" Bar. Net Section 1.56 □", Weight 5.31 lbs. per ft.

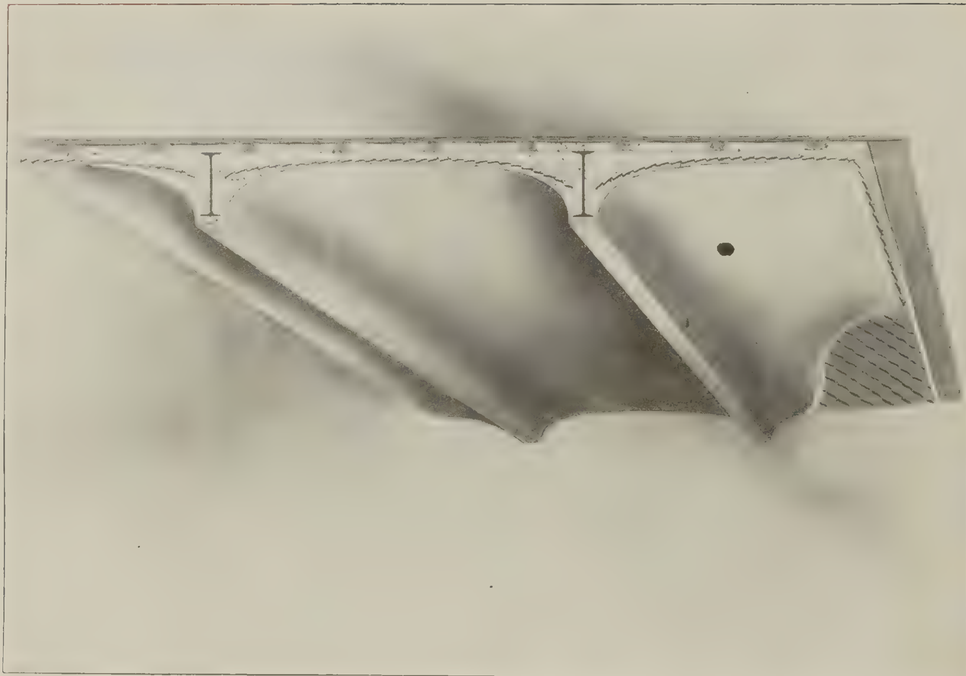
New Style Corrugated Bars.
A variation in weight of 5% either way is required.



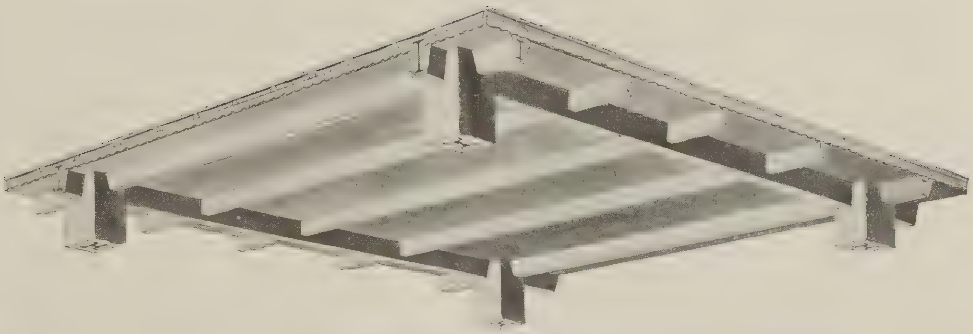
System No. 3.—Flat Slab Floor—For designing tables, see pages 128 and 131—Suitable for spans up to sixteen feet.



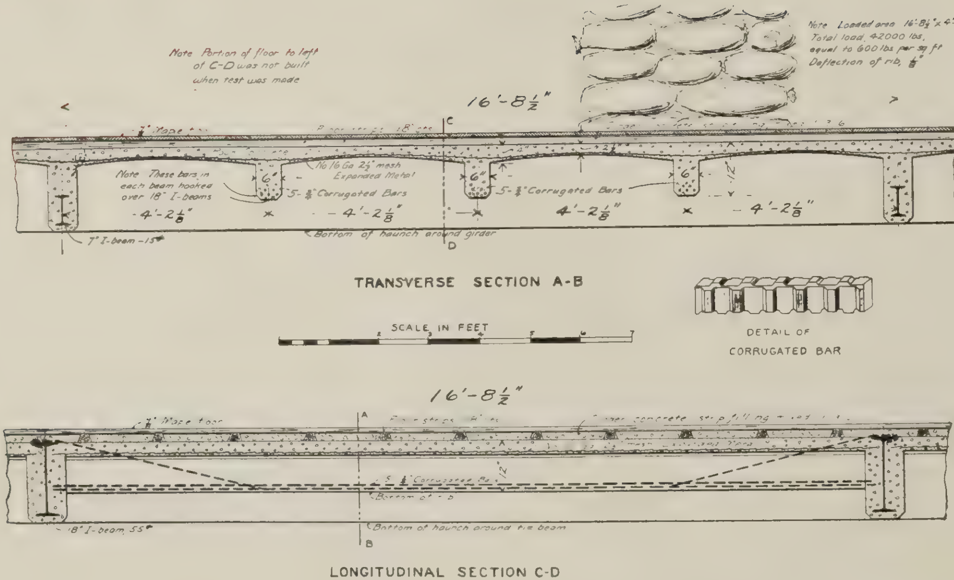
System No. 4.—Expanded Metal Flat Slab—For designing tables, see pages 126, 127, 129 and 130—Suitable for spans up to eight feet.



System No. 5.—Expanded Metal Flat Arch—Suitable for spans up to ten feet -No tie rods necessary.

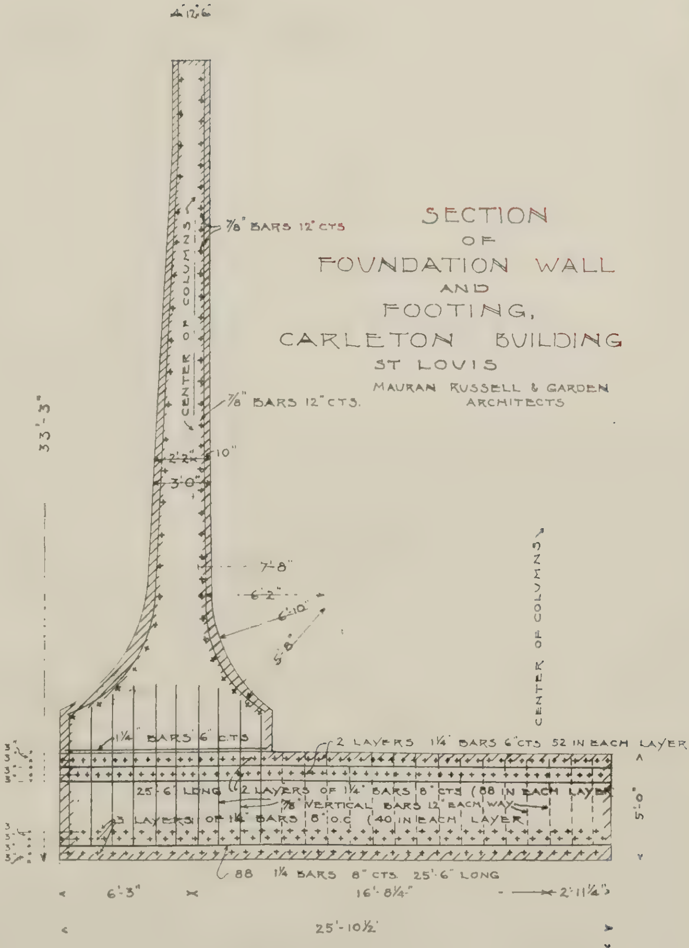


System No. 6.—Long Span Tee System Using Corrugated Bars in the Ribs and Expanded Metal in the Flat Slab. For designing table in good rock concrete, see page 143.



System No. 6.—Tee Floor—For designing table, see page 143.

Test on System No. 6, as shown on page 18. Rock Concrete, 1:2:5; Age 6 weeks. Load 600 pounds per square foot. Deflection at center of rib ⅛″.



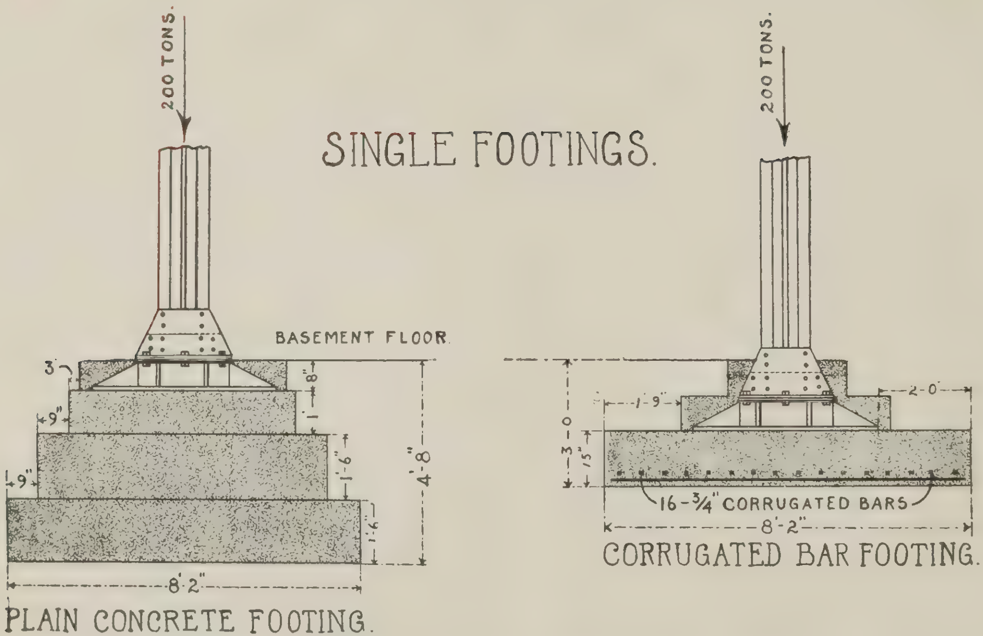
COLUMN FOOTINGS No 83-84-85-86-87-88.

J. R. Broderick, Engr.

Carleton Building—Completed Retaining Wall.

Interborough Power House, Rapid Transit R. R., New York.
S. L. F. Deyo, Chf. Engr.;
A. Van Vleck, Mech. and Const. Engr.
Bars used in Engine Foundations.

The tall building is the Wall St. Exchange Building, New York.
Clinton & Russell, Archts. Geo. A. Fuller & Co., Contrs.
Corydon Y. Purdy, Chief Engineer.
Bars used in cellar floor to resist upward water pressure.



Comparison between Plain and Reinforced Single Footings.

COMPARISON OF COST OF SINGLE FOOTINGS

PLAIN CONCRETE FOOTING

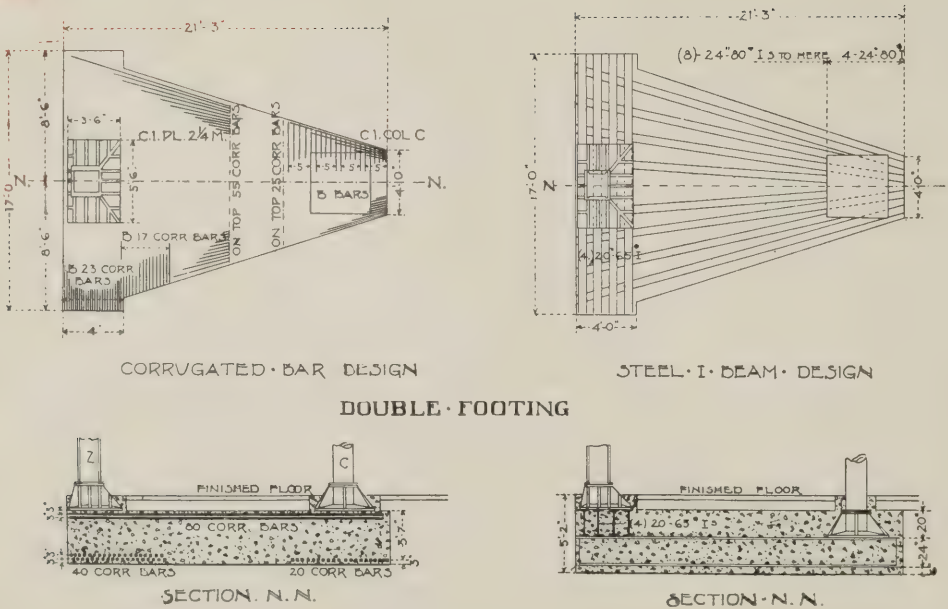
Excavation, 11½ cu. yds., @ 50c	$ 5.75
Concrete, 205 cu. ft., @ 20c	41.00
Total	$46.75

CORRUGATED BAR FOOTING

Excavation, 7½ cu. yds., @ 50c	$ 3.75
Concrete, 102 cu. ft., @ 20c	20.40
Corrugated Bars, 382 lbs., @ 3c	11.46
Extra column length, 85 lbs., @ 3½c	2.98
Total	$38.59

This shows that even in single piers a distinct saving is made by the reinforced concrete design. The percentage of saving increases with the size of the footing.

The chief recommendation of this construction, however, lies not so much in the decreased cost as in the greatly increased reliability. The plain footing depends upon the tensile strength of the concrete to give the required spread. No more unreliable factor of strength exists in the whole realm of building materials. In the corrugated bar design, even if the tensile strength of the concrete were zero, the strength of the footing would not be materially altered.



Comparison between Corrugated Bar and I Beam Double Footings.
Corrugated Bar Design used for the Norvell-Shapleigh Building, St. Louis.
Weber & Groves, Archts.

DOUBLE OR COMBINED FOOTINGS

On the foregoing page is shown a comparison between a Corrugated Bar and an I Beam footing, of equal strength, for two columns. The column to the left carries 358 tons, the other 222 tons. The area of the footing is 232 square feet, making an average pressure of 2.5 tons per square foot. The center of gravity of footing does not coincide with the resultant of the loads, resulting in a variation in soil pressure, which can be obtained by Hooke's law for beams $f=\frac{My_1}{I}$ where f is the increase or decrease in pressure in tons per square foot at the edge of the footing; y_1 the distance in feet from the edge in question to the center of gravity of footing; M is the revolving moment in foot tons around this center of gravity; and I is the moment of inertia of the footing plan in feet. In the case shown, I=7565. M=580x0.42=243.5 foot tons. From the small end to the center of gravity is 12.92′. This gives f_1=0.42 tons per square foot. In the same way f_2 is found to be 0.27 tons per square foot. Hence under one edge we have a pressure of 2.77 tons per square foot and under the other 2.08.

The maximum bending moment occurs at the point of zero shear and is 22,800,000 inch pounds for a width of 11.77 feet. Taking a factor of safety of four, we have an ultimate moment for a width of 1′ of 7,760,000 inch pounds. From formulae on page 88, for average concrete, this gives a thickness of concrete of 45″, and 3½ square inches of metal per foot of width=6, ⅞″ corrugated bars.

For the I-beam footing, the moment of 1,900,000 foot pounds requires 8, 24″—80-lb. beams.

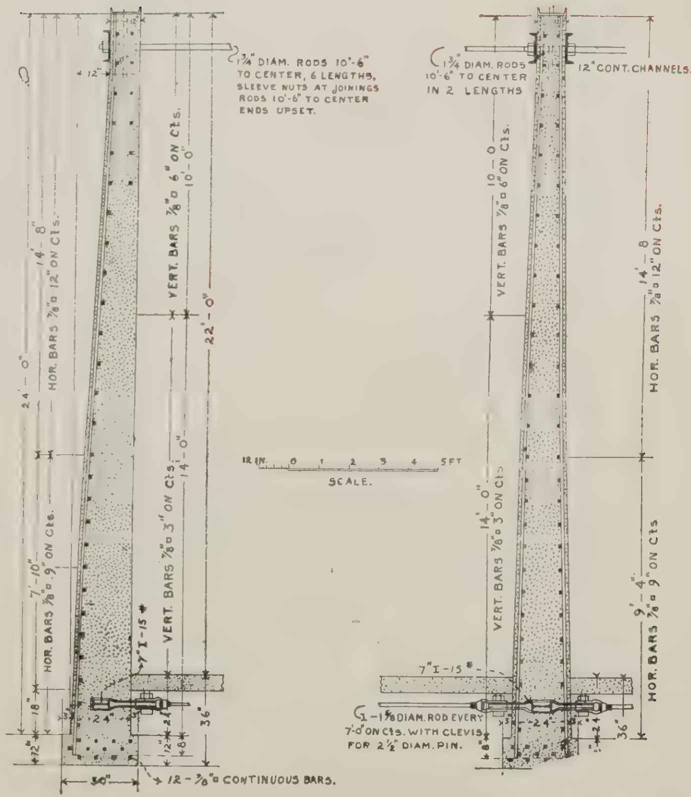
COMPARISON OF COST.

Corrugated Bar Footing.

Excavation, 39 cu. yds., @ 50c....	$ 19.50
Concrete, 870 cu. ft., @ 20c........	174.00
Bars, 4,106 lbs., @ 3c..............	123.18
Total..........................	$316.68

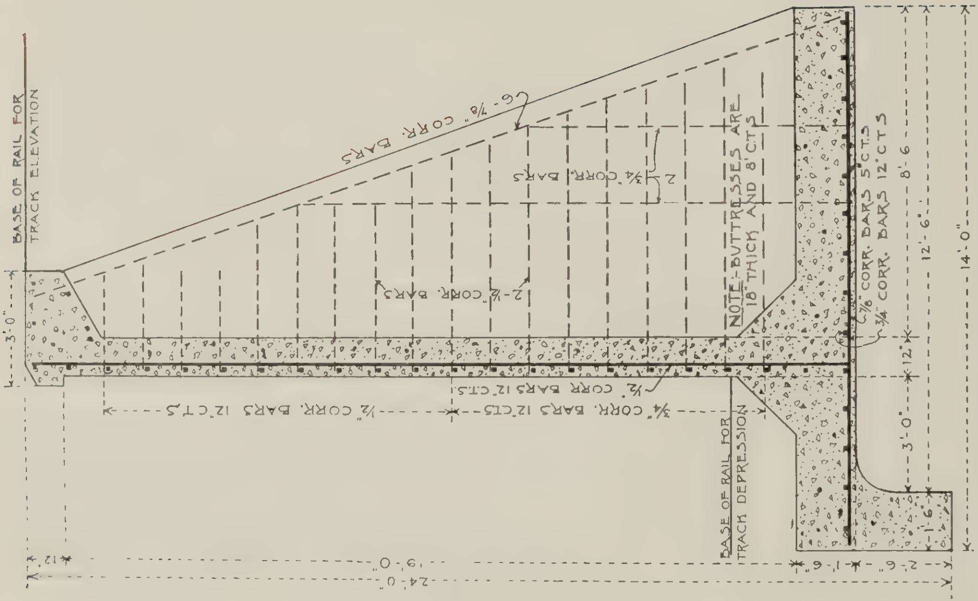
I-Beam Footing.

Excavation, 45 cu. yds., @ 50c....	$ 22.50
Concrete, 966 cu. ft., @ 20c........	193.20
Steel beams, 16,660 lbs., @ 2½c....	416.50
Bolts and sep's, 1,120 lbs., @ 2c....	22.40
Total..........................	$654.60



St. Louis Portland Cement Co., J. C. Robinson, M. E., Manager.
Section of Walls of Stockhouse.

St. Louis Portland Cement Co. Stockhouse, During Construction.



Retaining Wall for Track Elevation or Depression.
No expansion joints needed.

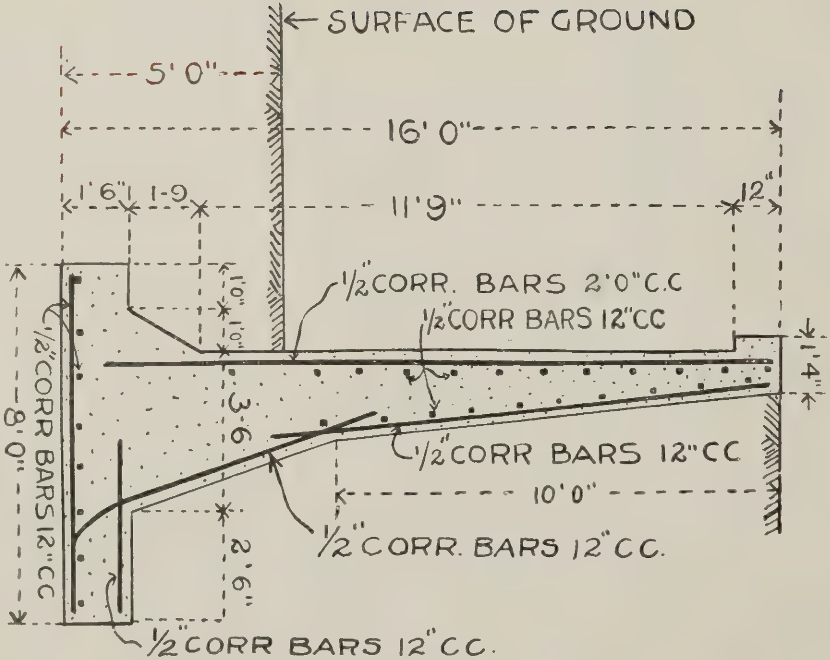
CONTINUOUS WALLS

One of the great advantages of reinforced concrete is in our ability to dispense with expansion joints in long structures. These may be built with the material in one piece from end to end, *a mile long if desired*, and by a properly proportioned longitudinal metal reinforcement, *shrinkage and temperature cracks can be entirely obviated.*

Most engineers have to be shown; and they will not believe it then unless they can see some scientific explanation of the matter. That explanation is as follows:

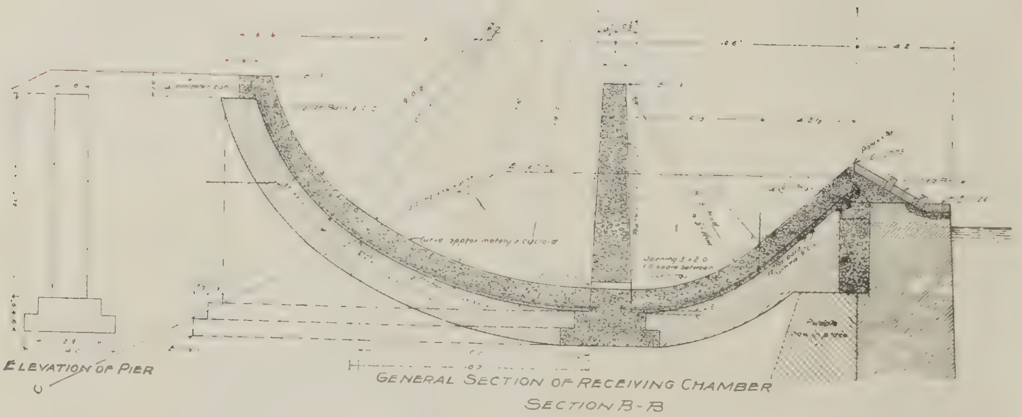
It has been shown by Considère, Hatt, and others, that concrete, when reinforced with metal well disseminated in small areas, will apparently stretch about ten times as much as when no metal is present, and that it will submit to proportionate elongations of about .0015. The co-efficient of expansion of concrete being .0000055, we find that *it would take a fall of 270° to develop a proportionate shortening equal to the wall's ability to stretch.* The wall will pull out in this manner at about three-fourths its full tensile strength, or say at 150 pounds per square inch.

The quantity of metal needed is enough to equal the tensile strength of the wall at an elongation of .0015, corresponding to a stress per square inch in the metal of 45,000 pounds. The area of metal would therefore be $\frac{1}{300}$ part of the area of the wall.

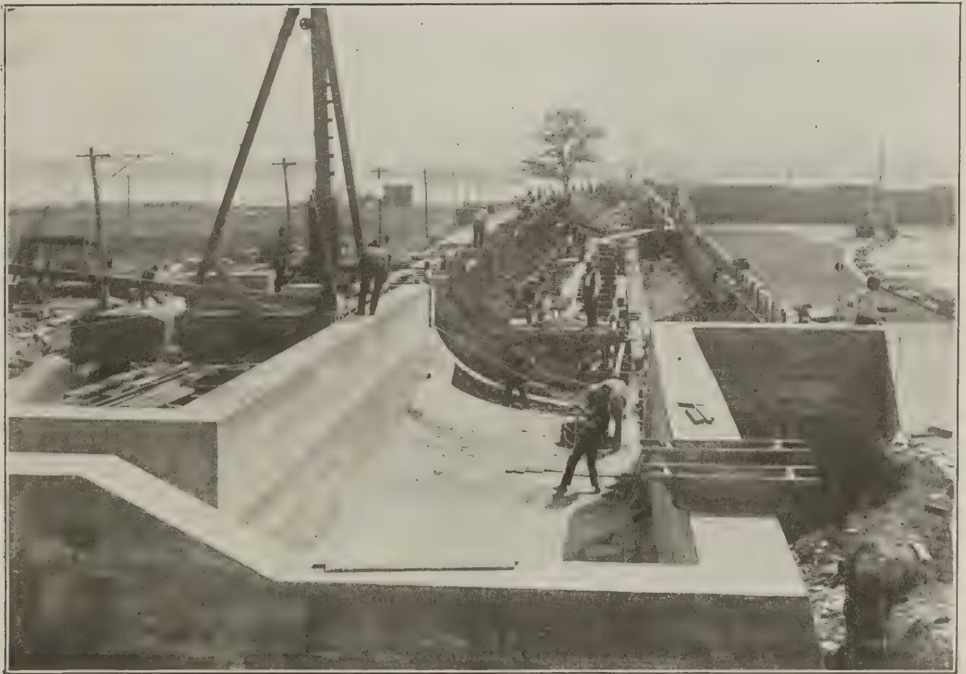


Section of Retaining Wall, Marion County, Ind.
H. W. Klausmann, County Surveyor.

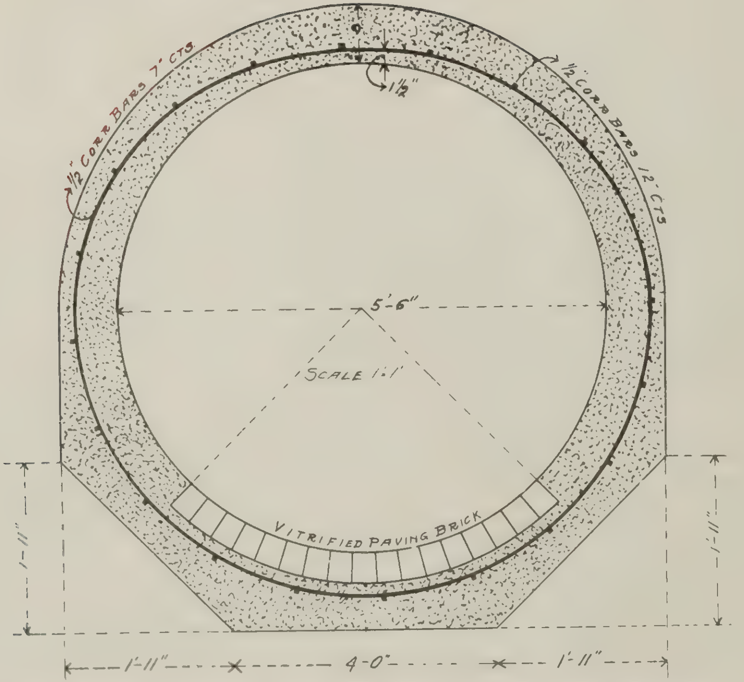
Retaining Wall, Marion County, Ind.



St. Louis Water Department—Section of Weirs.
B. C. Adkins, Water Commissioner.
E. E. Wall, Prin. Asst. Engr.

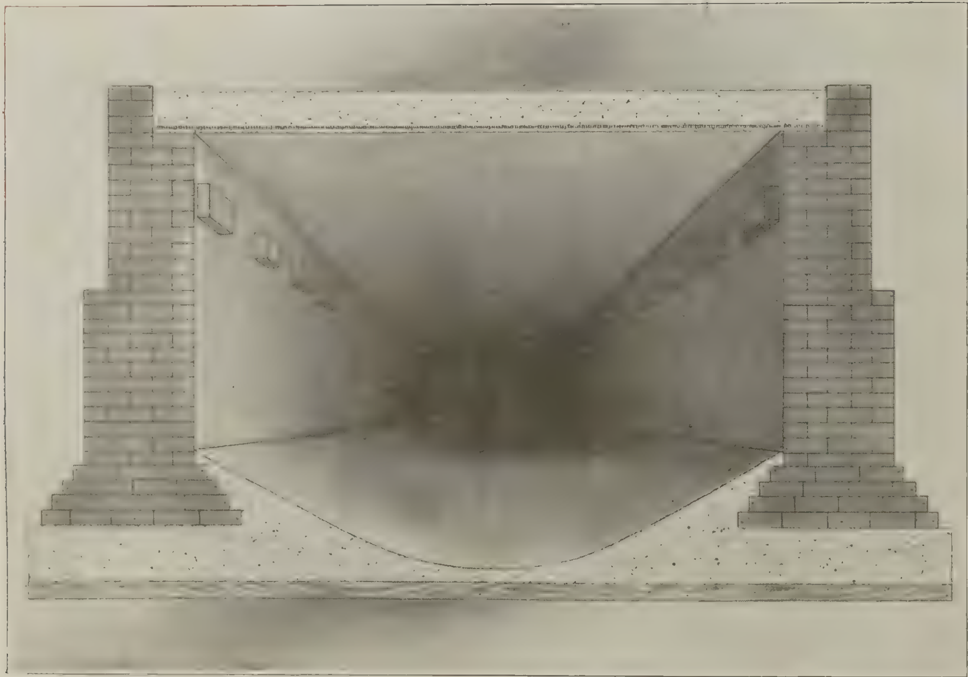


St. Louis Water Department—Weirs under Construction.



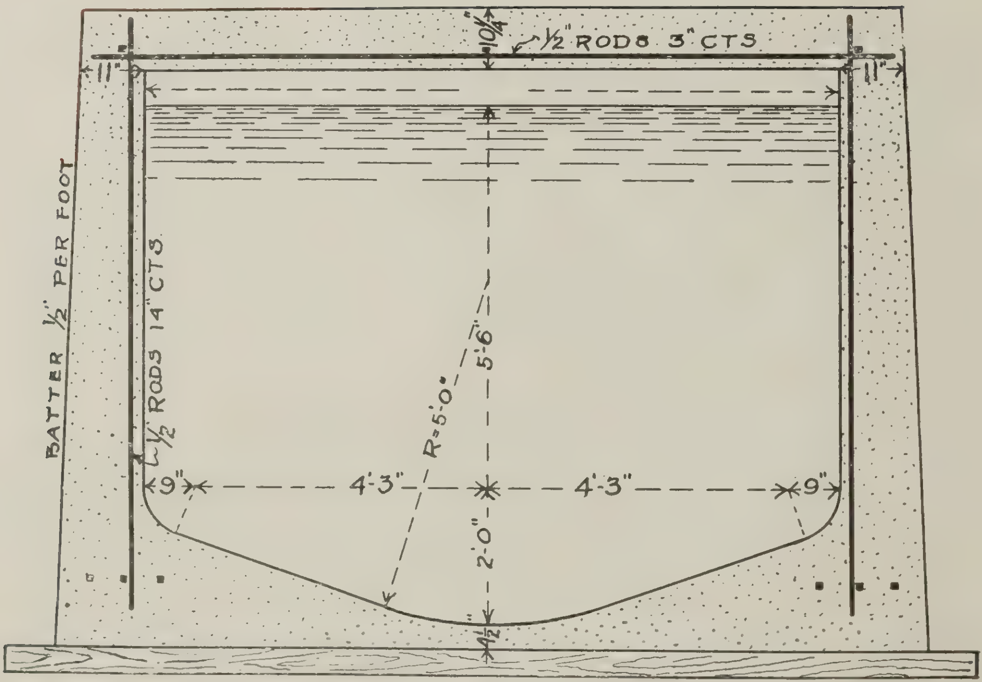
Cross Section of Conduit at Del Rio, Texas. J. W. Maxey, Engineer.

Del Rio Conduit under Construction.



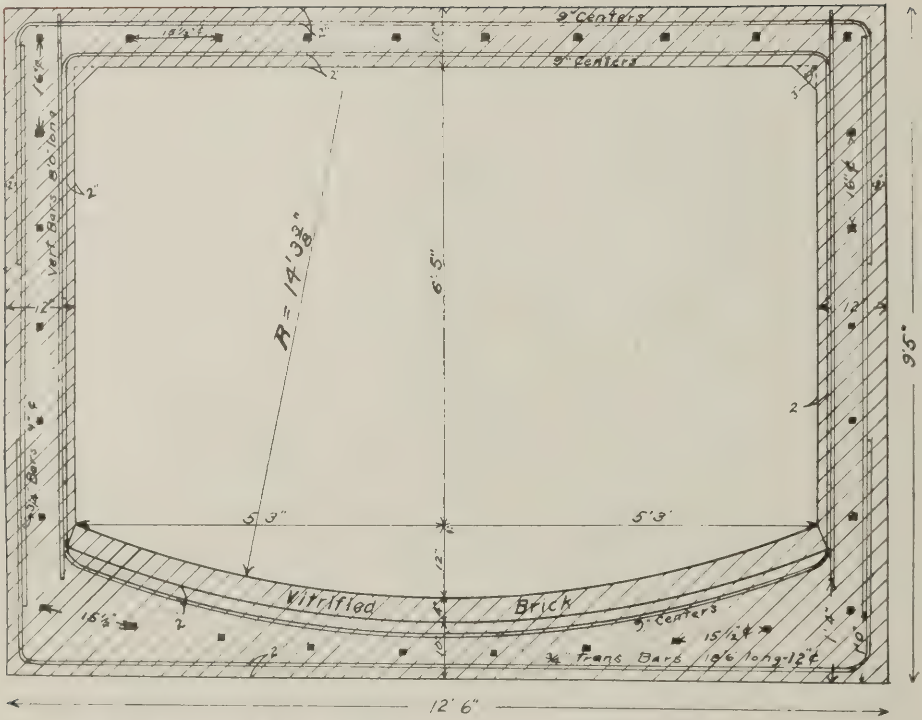
Section of New Orleans Drainage Canal. Maj. B. M. Harrod, Chief Engineer.

New Orleans Drainage Canal, Showing Test. Gravel Concrete 1:3:6; span 13′; slab 11¼″ thick; reinforcement ½″ ⊏ corrugated bars, 4¾″ cts.; load 51150 pounds on two 8″x8″ supports in center, 6 feet apart. Deflection scarcely appreciable.



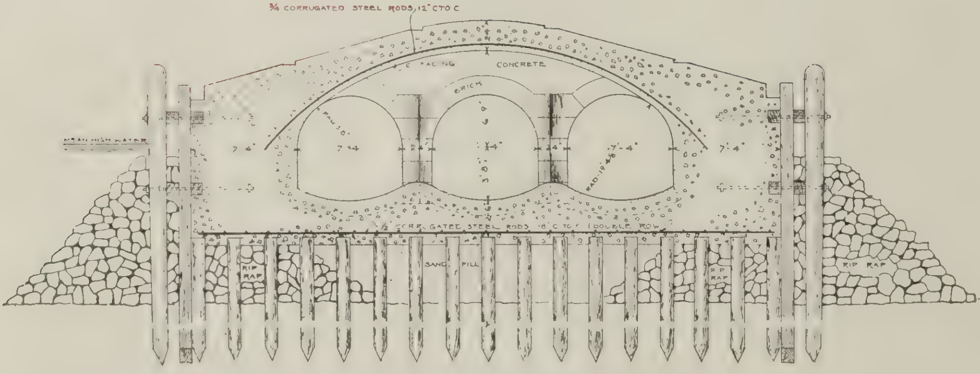
Last Type of New Orleans Drainage Canal.

Last Type of New Orleans Drainage Canal under Construction.



St. Louis Terminal Railway Association—Section of Sewer under Baggage Floor.
J. L. Armstrong, Engr. M. of W. A. P. Greensfelder, Asst. Engr.

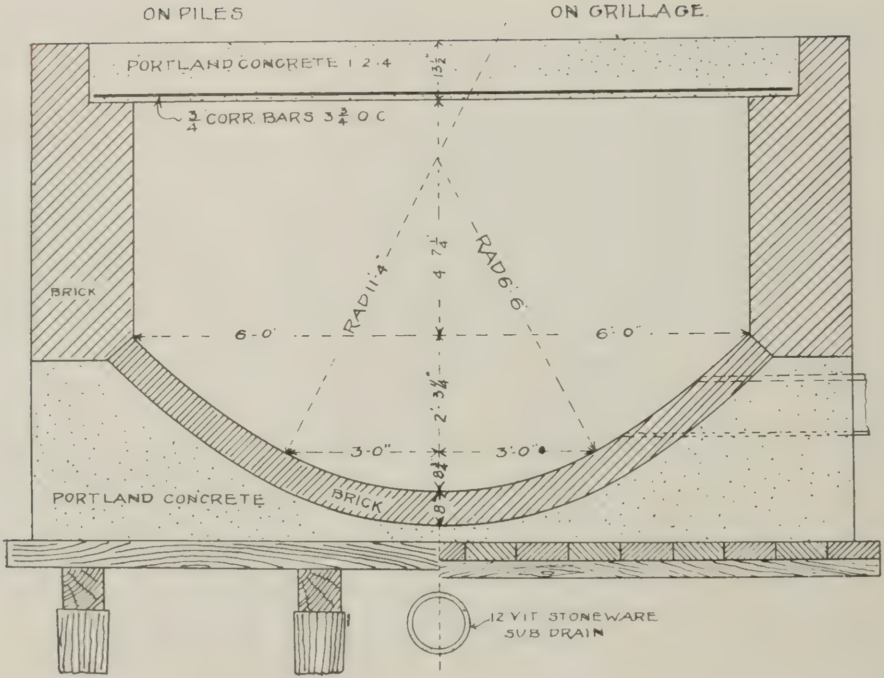
St. Louis Terminal Railroad Association—Meeting Point of Two Branches of Sewer.



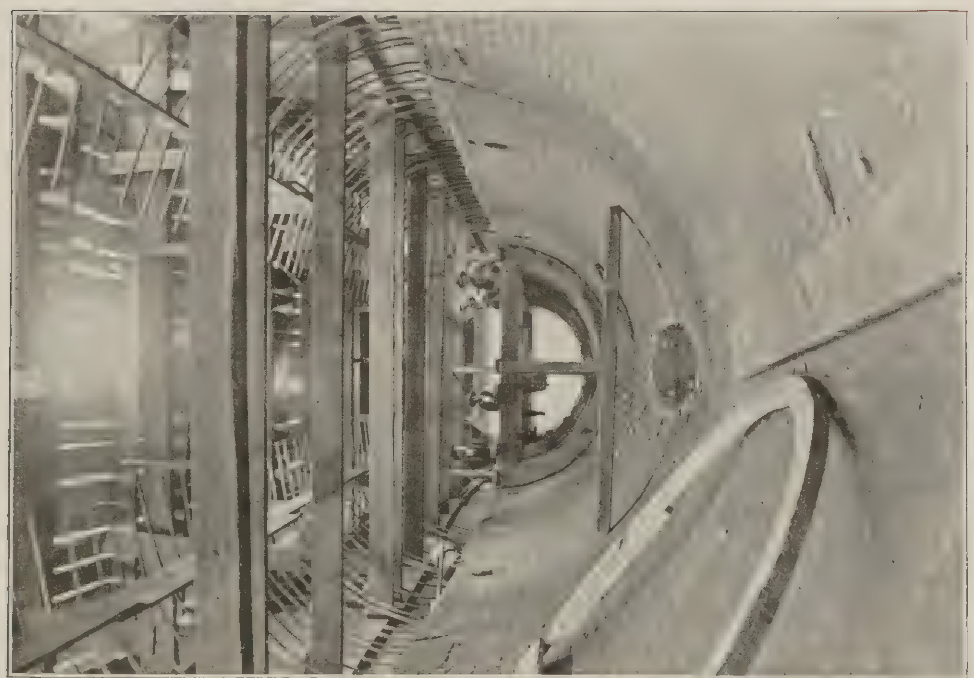
SECTION MAIN OUTLET SEWER BROOKLYN NEW YORK.

R. H. Asserson, Chf. Engr.

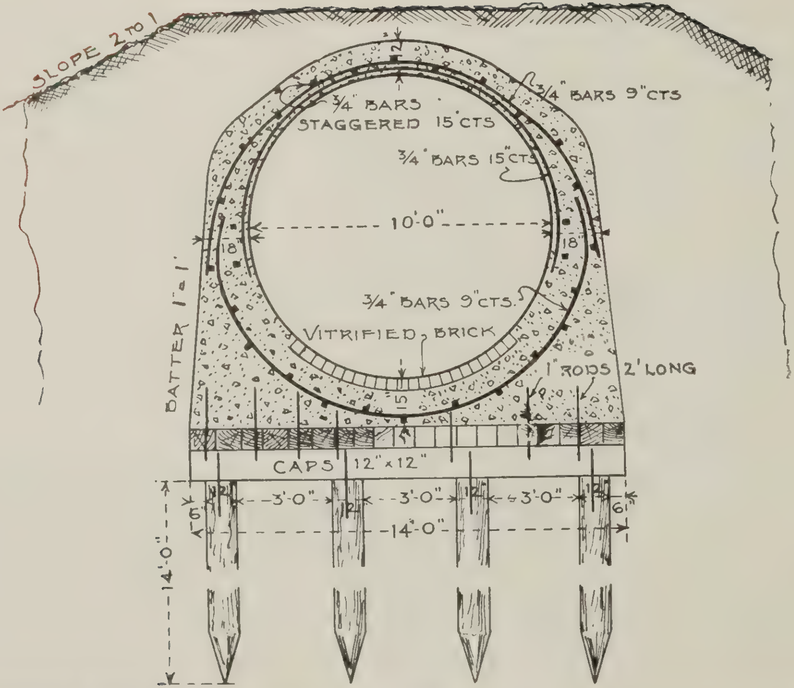
Main Outlet Sewer. Brooklyn, during Construction.



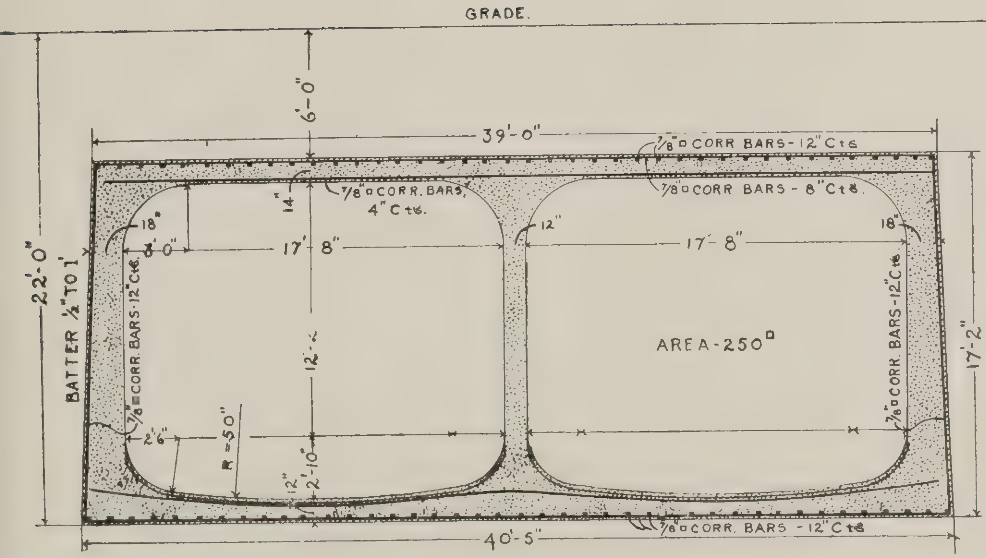
R. H. Asserson, Chf. Engr.



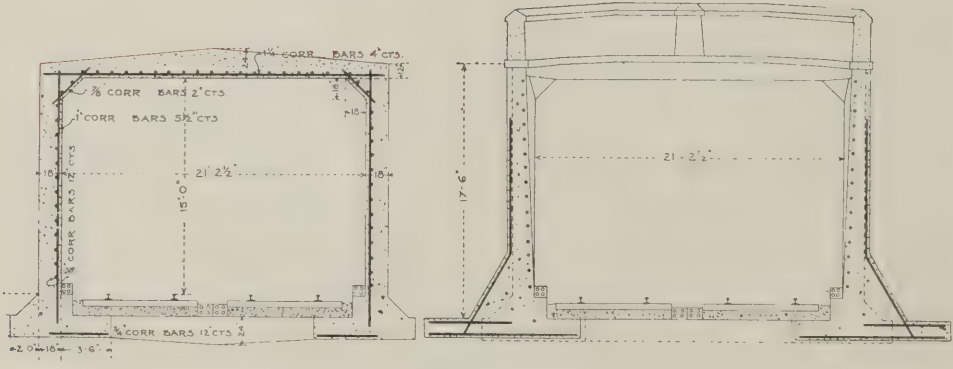
Another Type of Brooklyn Sewer Construction.



Proposed Section of Main Outlet Sewer, Kansas City, Mo. D. W. Pike, City Engineer.

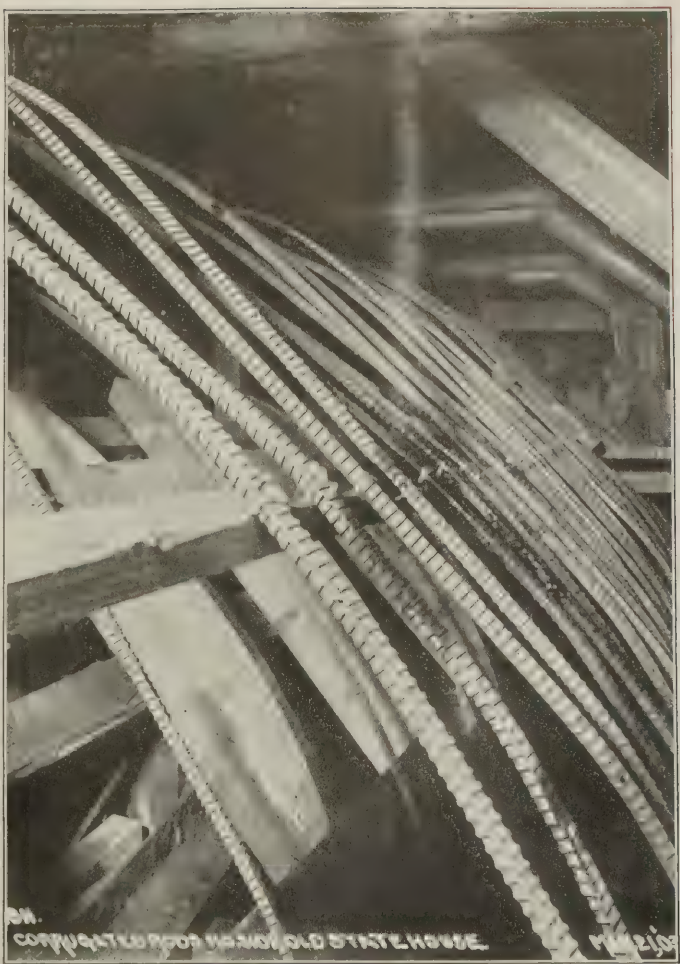


Proposed River des Pêres Sewer through Catlin Tract. Julius Pitzman, Engineer.



Section of Tunnel and Retaining Wall, Metropolitan Street Railway Co., Kansas City, Mo.
Ford, Bacon & Davis, Engineers.

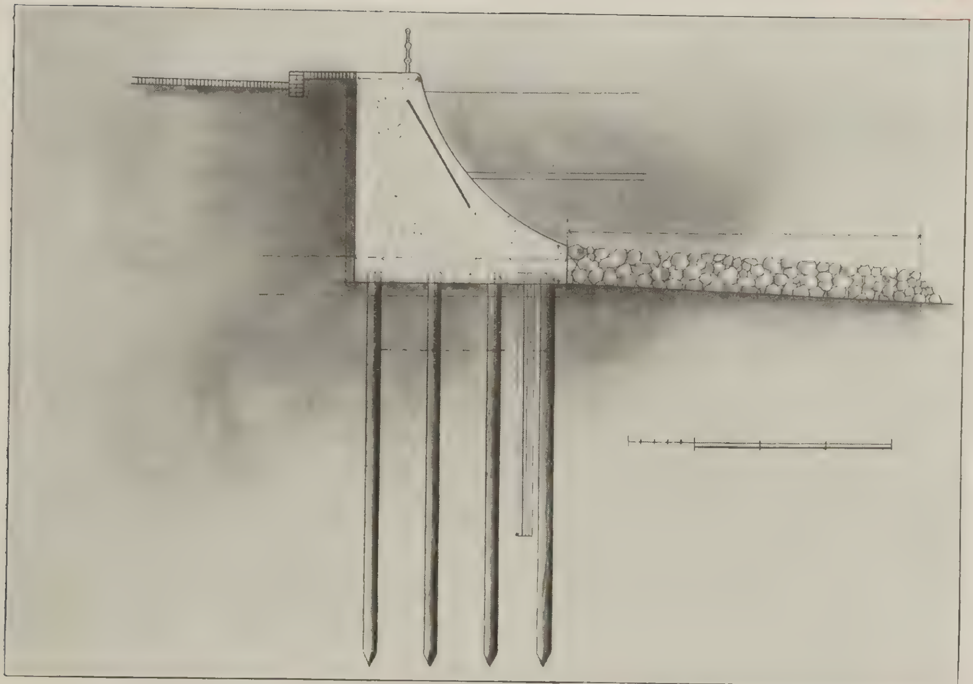
Metropolitan Street Railway Company Tunnel.



Tunnel, Boston Rapid Transit Commission. Howard A. Carson, Chief Engineer,

Boston Rapid Transit Subway. Howard A. Carson, Chief Engineer.

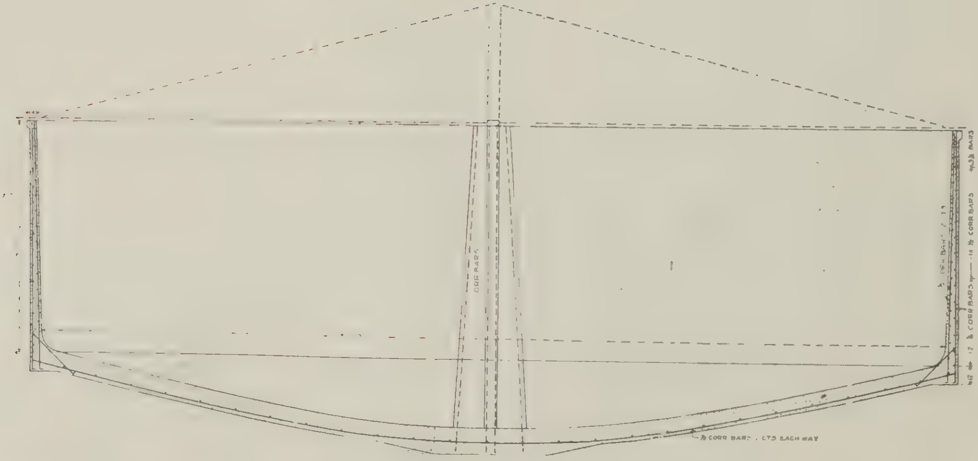
Missouri Pacific R. R. Grain Bins at Kansas City. Metcalf & Metcalf, Engrs.



Galveston Sea Wall. Geo. W. Boschke, Engr. of Constr.

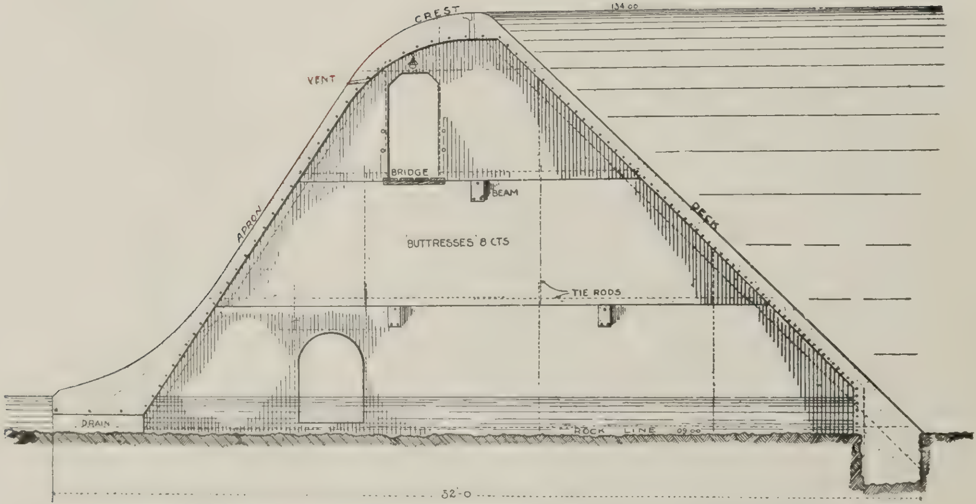
Galveston Sea Wall during Construction.

Galveston Sea Wall—Bird's-eye View.

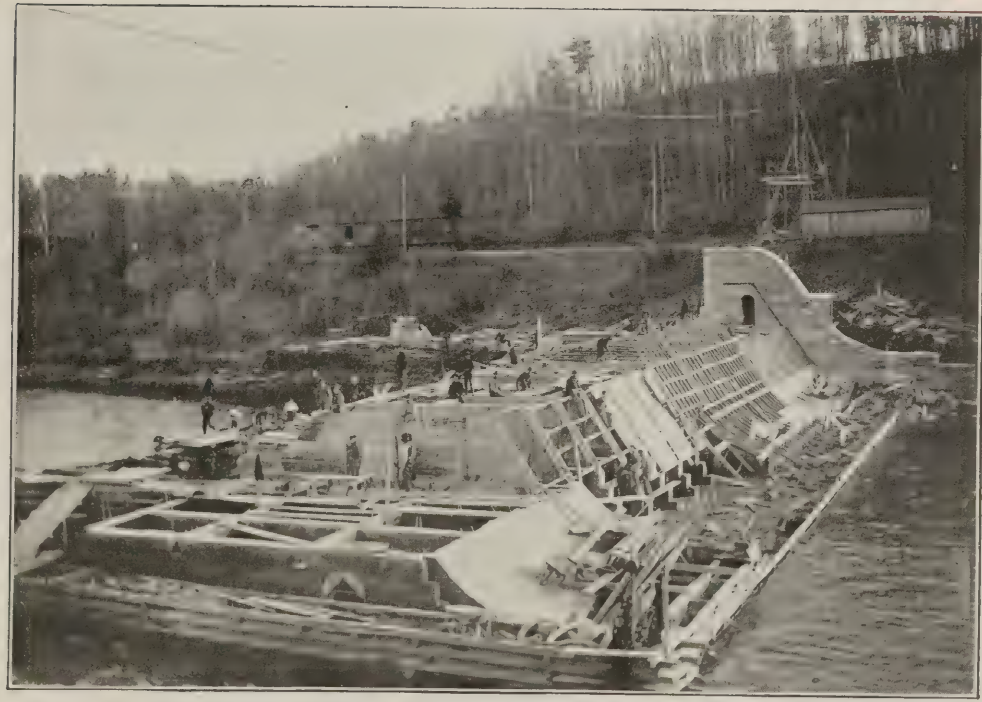


Reservoir at Lake Geneva, Wis. A. C. Warren, Engr.

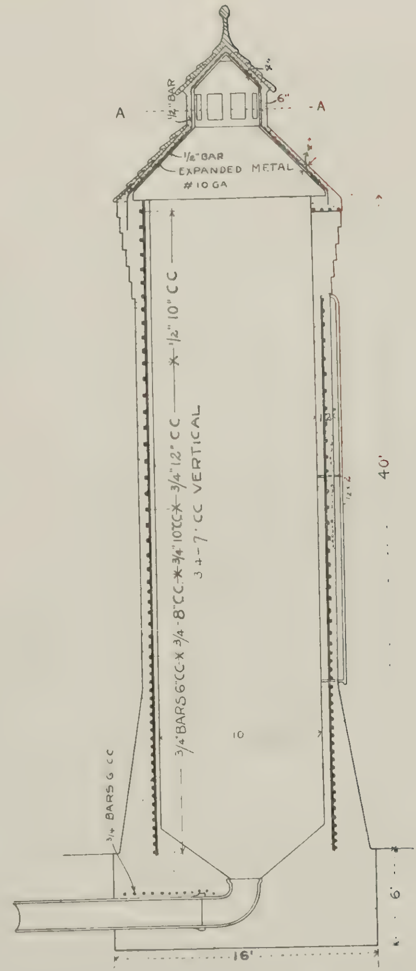
Photograph of Completed Lake Geneva Reservoir.



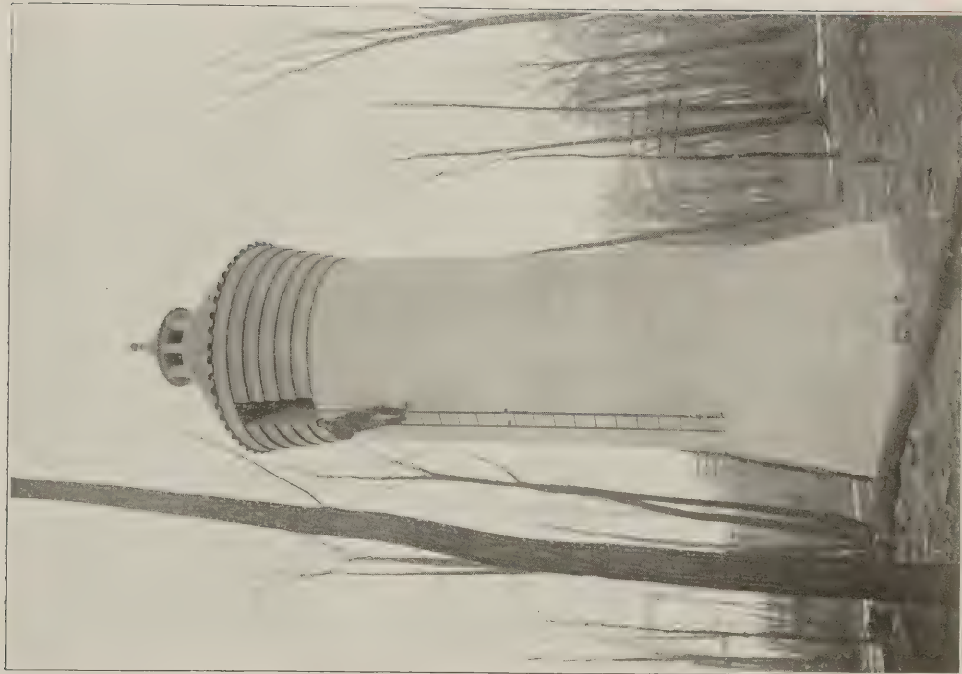
Reinforced Concrete Dam Across the Battenkill.
Built for the American Wood Board Co., Schuylerville, N. Y.
Patented by Ambursen Hydraulic Construction Co., Boston, Mass.



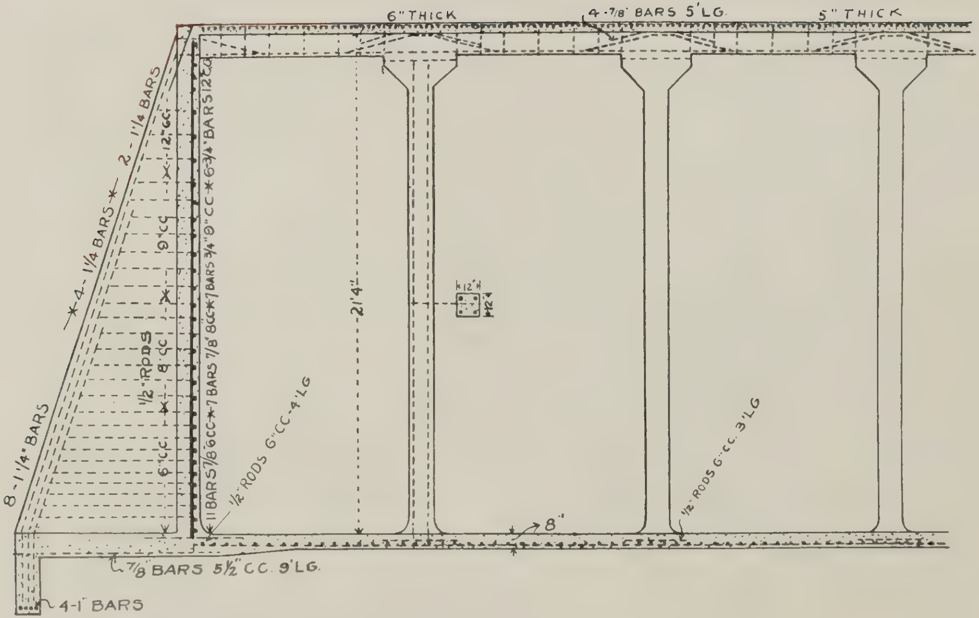
Ambursen Dam at Schuylerville under Construction.



Water Tower for Water Works at East Orange, N. J.
C. C. Vermeule, Conslt. Engr.
Commonwealth Roofing Co., Contrs.

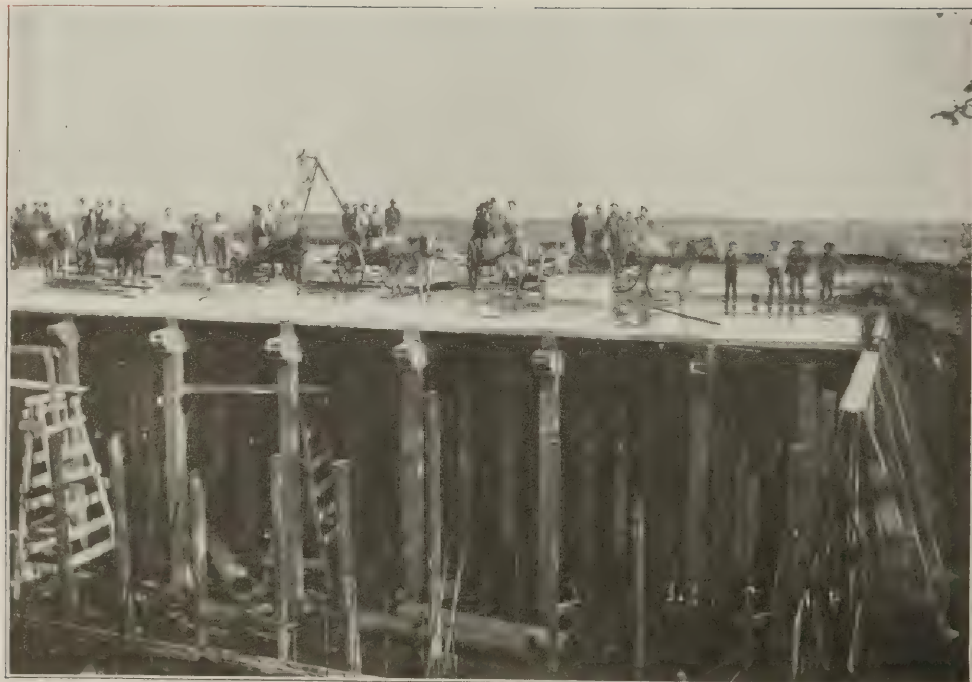


Photograph of Completed Tower.

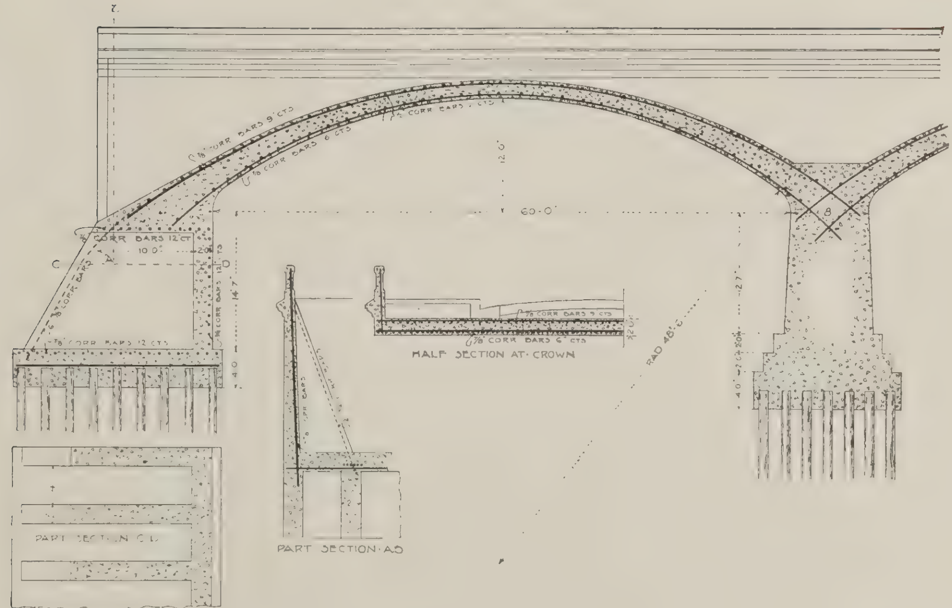


Section of Reservoir Construction, East Orange, N. J.
C. C. Vermeule, Conslt. Engr.
Commonwealth Roofing Co., Contrs.

East Orange Reservoir under Construction.

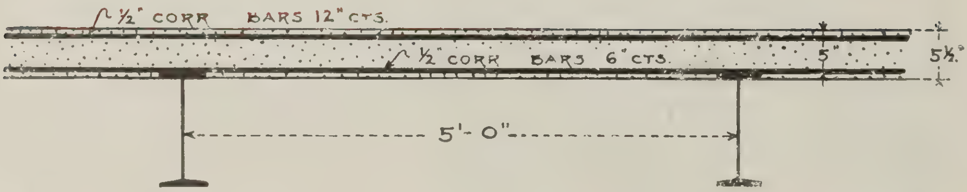


East Orange Reservoir under Construction.



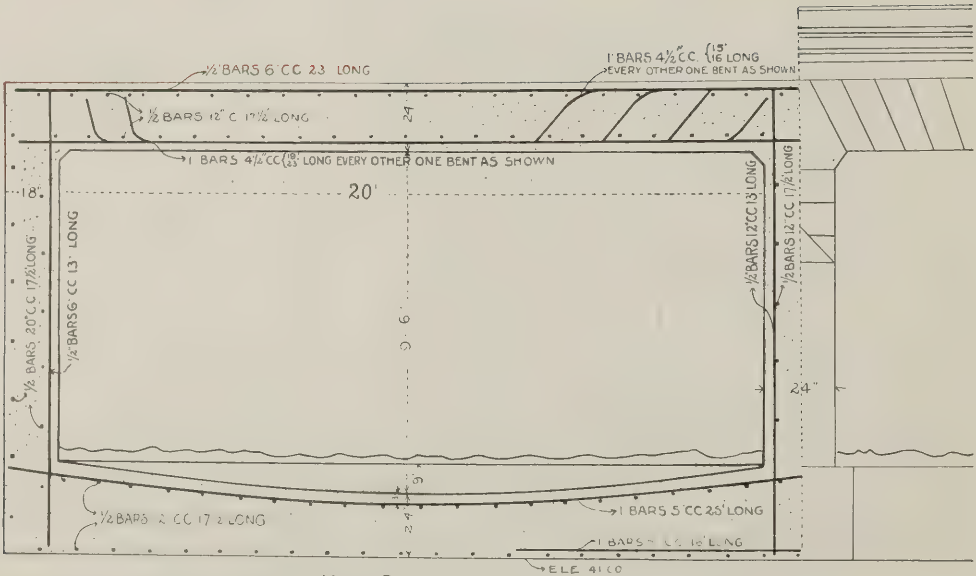
Arch with Hollow Abutments.

ST. LOUIS EXPANDED METAL FIRE PROOFING CO.



Cross Section of Highway Bridge Floor Construction. Designed for Cooper's Class A specifications. Many floors like this have been built.

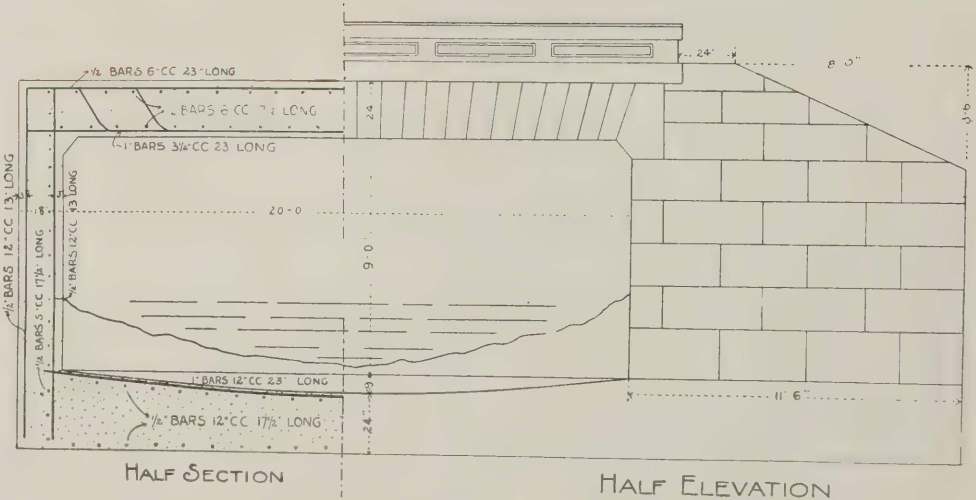
Expanded Metal Floor Construction on Highway Bridge at Waco, Texas. Span 535 feet.



HALF SECTION

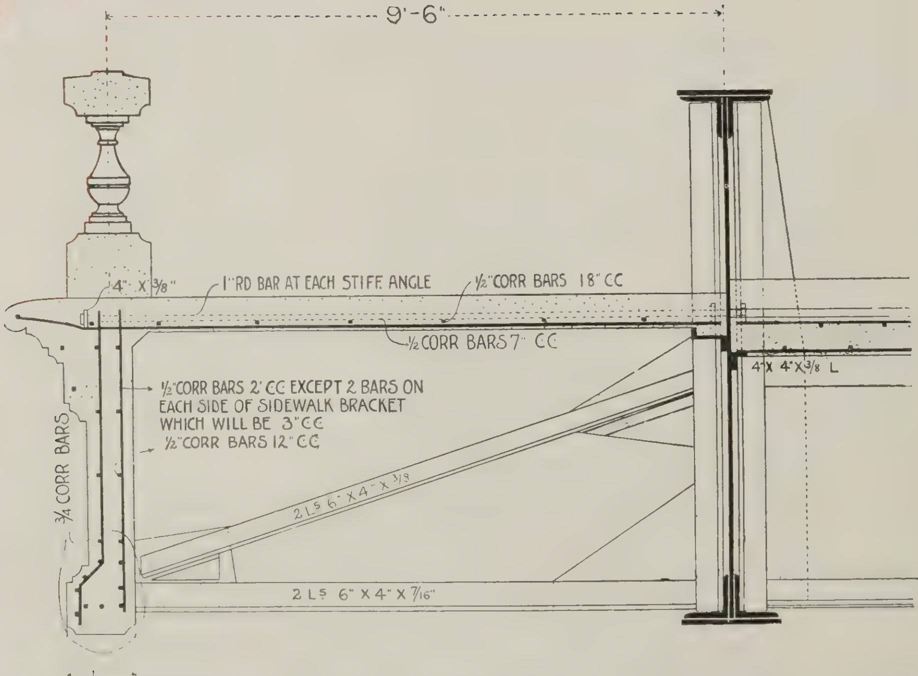
Section of Highway Culvert Construction, Marion Co., Ind.
H. W. Klausmann, County Engr.

Completed Culvert, Marion Co., Ind.



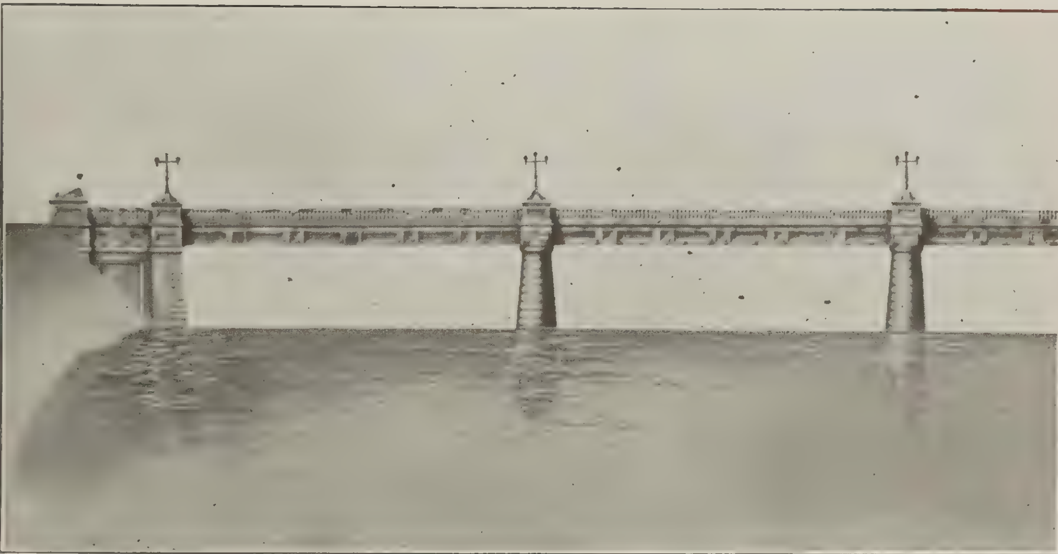
Section of Highway Culvert Construction. Marion Co., Ind.
H. W. Klausmann, County Engr.

Completed Culvert, Marion Co., Ind.

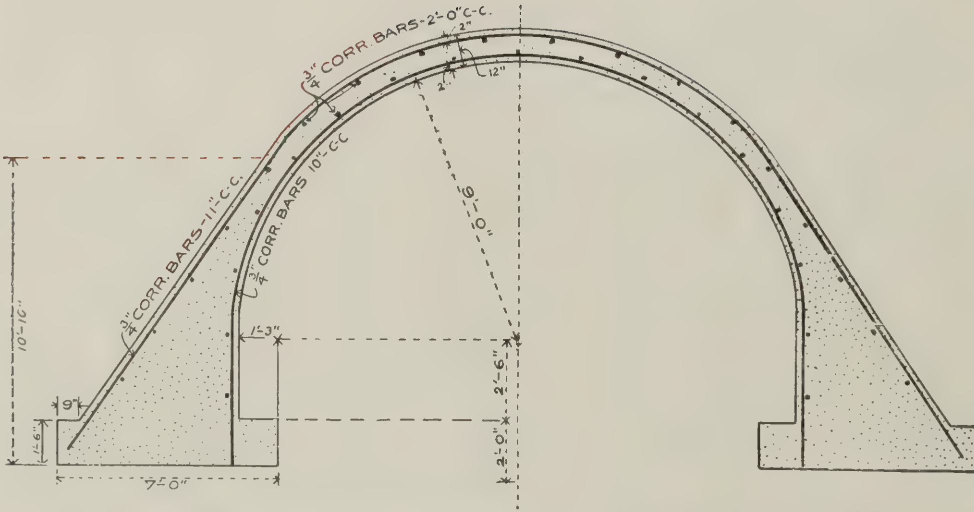


Section showing Ornamental Balustrade of Reinforced Concrete Screening Steel Plate Girder, Indianapolis, Ind.

H. W. Klausmann, County Engr.

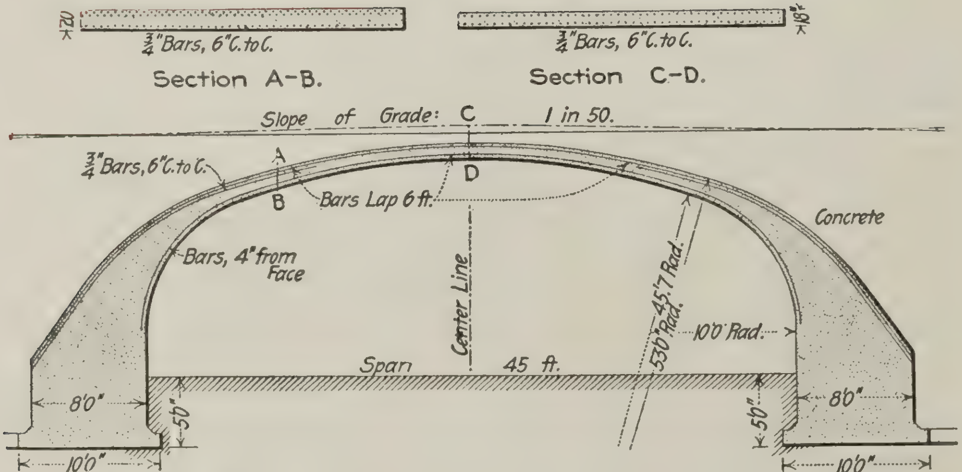


Photograph of Water Color Drawing, showing one-half of Completed Plate Girder Bridge with Ornamental Balustrade Screen. This Bridge is now under Construction.



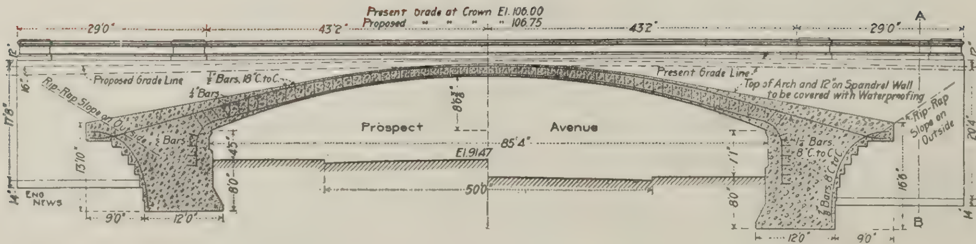
Section of Highway Culvert at South Bend, Ind. A. J. Hammond, City Engr.

Completed Culvert, South Bend, Ind.



River des Pêres Arch, Forest Park, St. Louis. R. H. Phillips, Chf. Engr.

River des Pêres Arch—Completed Structure.

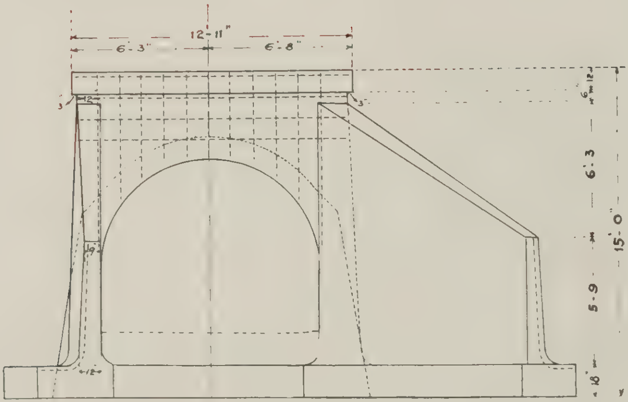


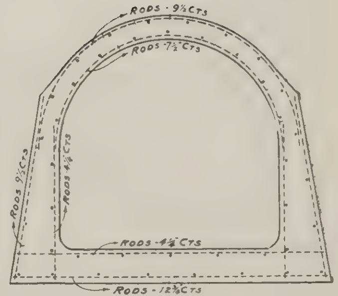
Seeley Street Bridge, Brooklyn, N. Y. G. W. Tillson, Chief Engineer;
E. J. Fort, Assistant Engineer.
D. Cuozzo & Bro., Contractors.

Seeley Street Bridge, Brooklyn, during Construction.

Seeley Street Bridge, Brooklyn, during Construction.

Completed Seeley Street Bridge, Brooklyn.

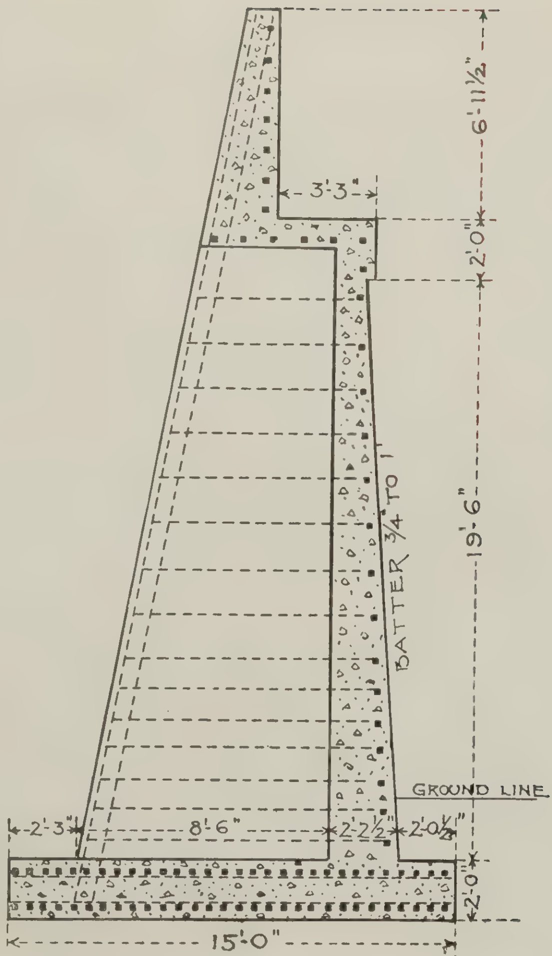




Section Showing Spacing Of Rods.

Section of Culvert on Wabash Railroad near Carpenter, Ill. W. S. Newhall, Chief Engineer; A. O. Cunningham, Bridge Engineer.

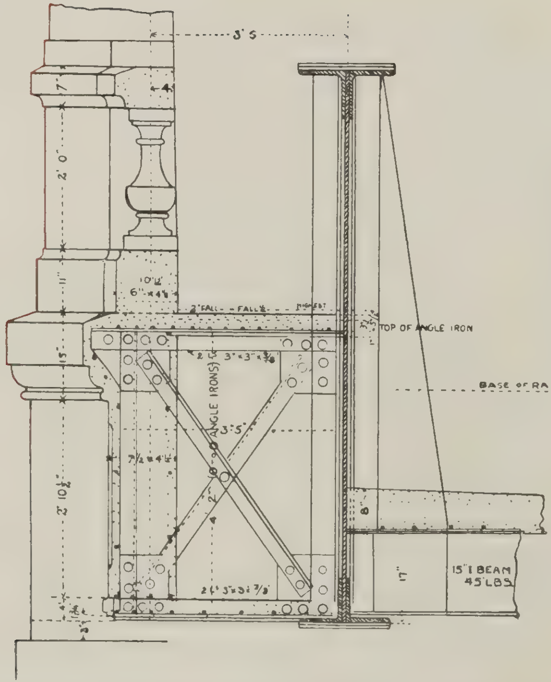
Wabash Culvert in Process of Construction.



Section of Abutment on Wabash Railroad at Monticello, Ill.
W. S. Newhall, Chief Engineer;
A. O. Cunningham, Bridge Engineer.

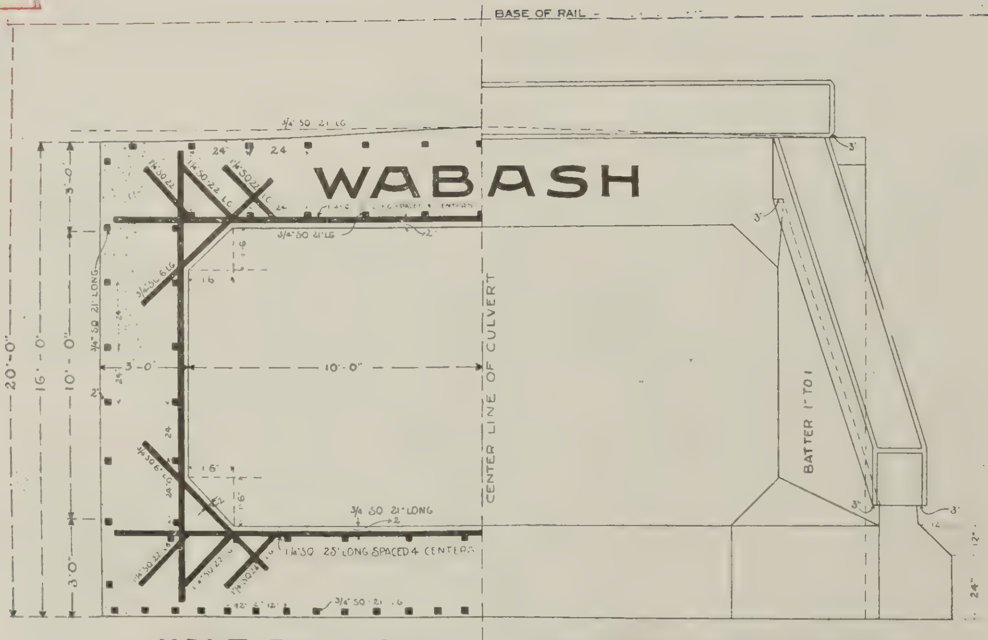
Monticello Bridge on Wabash Railroad. Photograph of Back of Abutment.

ST. LOUIS EXPANDED METAL FIRE PROOFING Co.



Wabash Plate Girder Bridge with Reinforced Concrete Floor, Hollow Abutments and Ornamental Balustrade, in Forest Park, St. Louis.
W. S. Newhall, Chf. Engr.;
A. O. Cunningham, Bridge Engr.

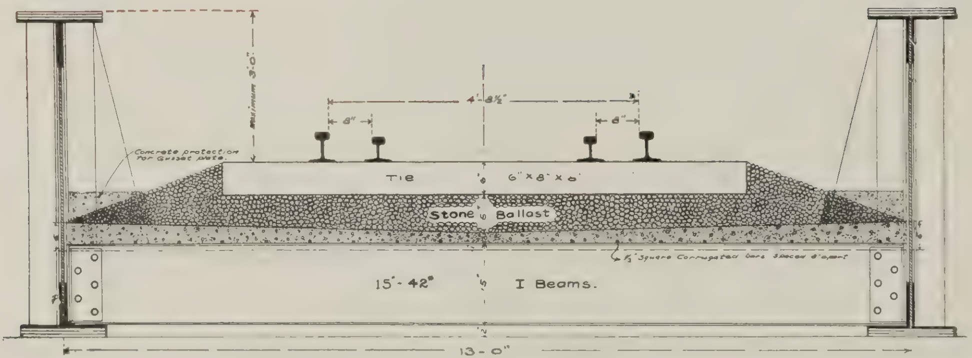
Completed Wabash Bridge, Forest Park, St. Louis.



HALF SECTION **HALF END ELEVATION**

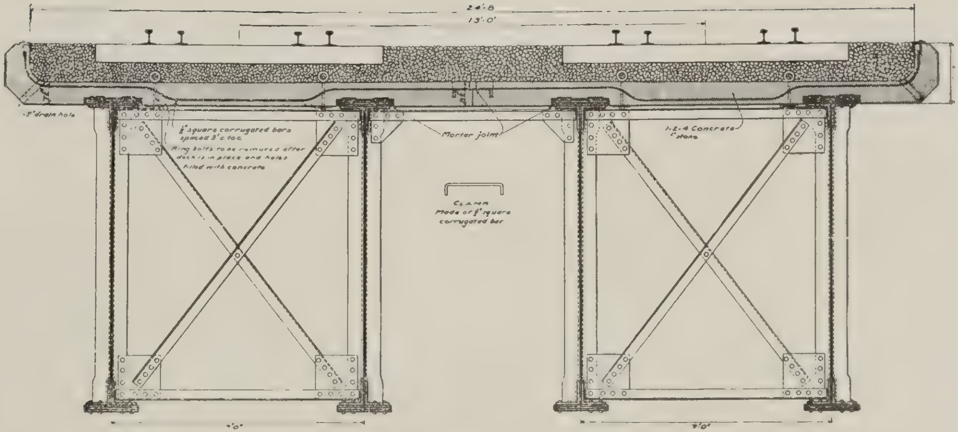
Section of Flat Top Culvert, 20′ Span, Wabash R. R., near St. Louis, Mo.
W. S. Newhall, Chf. Engr.;
A. O. Cunningham, Bridge Engr.

Completed 20′ Culvert, Wabash R. R.

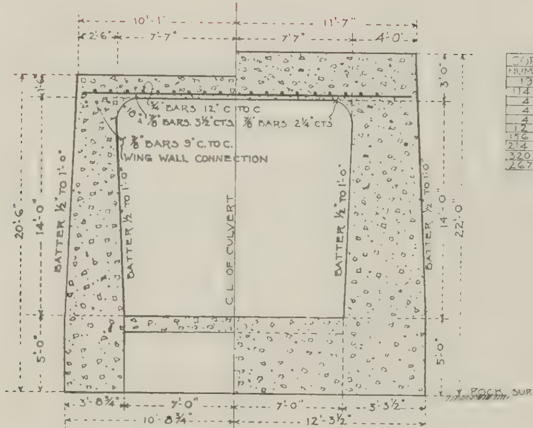


-Section-

One Type of Solid Reinforced Concrete Bridge Floor, Wabash Railroad.
W. S. Newhall, Chf. Engr.;
A. O. Cunningham, Bridge Engr.



One Type of Solid Reinforced Concrete Bridge Floor on the C., B. & Q. R. R.
W. L. Breckinridge, Chief Engineer; C. H. Cartlidge, Bridge Engineer.



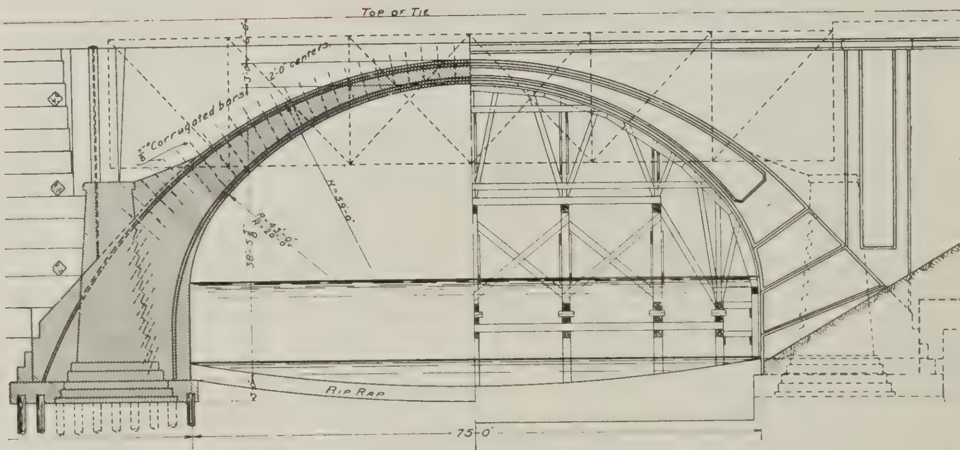
SECTION · AT · END SECTION · AT · CENTER

MATERIAL · REQUIRED

CORRUGATED · STEEL · BARS				CONCRETE - 1980.5 CU YDS
NUMBER	SIZE	LENGTH		1782 CU. YDS. STONE
19	¾	27'-6"		891 " " SAND
114	¾"	28' 6"		2139 Bbls CEMENT
4	¾"	26'-6"		
4	¾	26'-0"		
4	¾"	25'-6"		
12	⅞	24'-9"		
156	3	19' 6"		
214	⅞	20' 6"		
320	⅞"	21'-6"		
267	⅞"	22' 6"		

C. B. & Q. RY.
Old Monroe to Mexico Branch
BRIDGE No. 77.22
OVER
CLEAR · BRANCH
CONCRETE STEEL BOX CULVERT

Clear Branch Culvert. Nearly 100 Culverts of this type built on the Burlington Road in the years 1903 and 1904.



Plano Arch, 75′ Span; C., B. & Q. R. R. W. L. Breckinridge, Chief Engineer;
C. H. Cartlidge, Bridge Engineer.

Completed Plano Arch.

Culvert on C., M. & St. P. R. R. C. F. Loweth, Engineer Bridges and Buildings.

Culvert on C., M. & St. P. R. R. C. F. Loweth, Engineer Bridges and Buildings.
Many Reinforced Concrete Culverts of all types have been built by this Road.

Culvert of 20′ Span, P. S. & N. R. R. M. F. Bonzano, Chief Engineer.

Culvert of 6′ Span, P. S. & N. R. R. M. F. Bonzano, Chief Engineer.
Many Culverts of both Arch and Box Sections built on this Road in the last two years.

Four-Track Reinforced Concrete Arch at Willoughby Run on L. S. & M. S. R. R. Clear Span, 154'.
E. A. Handy, Chf. Engr.;
Frank Beckwith, Engr. of Bridges.

Willoughby Run Arch Completed.

Photograph of Lake Shore 30′ Arch during Construction.

Reinforced Concrete Arch, 30′ Span, L. S. & M. S. R. R. E. A. Handy, Chief Engineer;
H. H. Ross, Assistant Engineer.

Angola Reinforced Concrete Arch, L. S. & M. S. R. R.
E. A. Handy, Chf. Engr.;
Frank Beckwith, Engr. of B. & S.

Apprcach to Bridge Across Mississippi River at Thebes, Ill. Noble & Modjeski, Engineers.

Reinforced Concrete Arch, 60′ Span, on the Illinois Central Railway. H. U. Wallace, Chief Engineer;
H. W. Parkhurst, Bridge Engineer.

Reinforced Concrete Arch, 75′ Span, on the Illinois Central Railway. H. U. Wallace, Chief Engineer;
H. W. Parkhurst, Bridge Engineer.

REINFORCED CONCRETE BEAMS

The position of the neutral axis in a reinforced beam is almost constantly changing. At the beginning of the loading it is at the center of gravity of the section transformed into its equivalent in concrete by building out wings on each side opposite the plane of reinforcement, having a depth equal to the thickness of the metal and a total area equal to the area of metal multiplied by the ratio of the modulus of elasticity of the steel to the original modulus of the concrete in tension. The neutral axis stays practically at this position until the stress on the extreme fibre of the concrete in tension equals its tensile strength. It then rises, as the loading proceeds, until the stress on the extreme fibre of the concrete in compression amounts to about one-half its ultimate strength, at which point the modulus of elasticity of the concrete in compression begins to decrease. This checks the upward movement of the axis, finally stopping it altogether sometime before the maximum load capacity of the beam is reached. It is the peripatetic movement of the neutral axis which makes it impossible to give, in simple equations, a satisfactory scientific expression for the conditions at all stages. Fortunately this is not necessary. We are chiefly interested in knowing how to design the most economical beam for a given strength.

Most formulæ for the strength of reinforced concrete beams are based upon a rectilinear relation between stress and strain, and the *safe* values inserted therein, instead of the *ultimate* values. In our judgment

this is not wise, as it is impossible to know what factor of safety is obtained. Most of these formulæ will take 16,000 pounds per square inch for the safe stress in the steel and say that there will be a factor of safety of four on the structure, because the ultimate strength of the steel is 64,000 pounds per square inch. But when the elastic limit of the metal is passed *its modulus drops from* 30,000,000 *to* 5,000,000 and the cracks in the concrete become so very large immediately that we do not consider as *available* any strength that can be obtained beyond this limit; though this excess is considerable if the quantity of reinforcement used is only *one-half* what it should be, as is the case in the method above described. With only one-third the quantity of metal necessary to develop the required ultimate strength at the elastic limit, it is possible to break the metal entirely in two. For example, in a six-inch slab of rock-concrete having expanded metal imbedded in its lower portion, the expanded metal will always be broken apart, though this is soft box-annealed material. But the factor of safety for such construction should be four on the elastic limit, which would be equivalent to about six on the maximum load. When, therefore, we give the beam credit for no more strength than it can develop at the elastic limit of the steel reinforcement, it is desirable that this limit should be fairly high. With an elastic limit of 30,000 pounds per square inch the most economical quantity of metal reinforcement is 1.5 per cent of the area of the concrete, while with a limit of 50,000, one per cent only is

required, or a saving of approximately one-third in the cost of the metal.

As has been stated in the introduction, page 5, there is still some discussion as to just when the first crack develops in reinforced concrete; but as also there shown, a proper reinforcement will cause the beam to develop a large number of cracks very close together, in which case these cracks will be of no material consequence so long as the bars are stressed inside the elastic limit. Corrugated bars will accomplish this result. The cracks will be close together, small in size, and will not be able to reach the bar itself. With plain bars, or bars of less positive form of bond, this is not true; and beams reinforced with such material cannot demonstrate immunity from injury so long as the stress in the bars is inside the elastic limit. Such beams exposed to the action of the atmosphere for a few months would be liable to have the reinforcement much corroded in time.

In the following discussion it is assumed that a section plane before bending is plane after bending up to an elastic limit in the metal reinforcement of 50,000 pounds per square inch, and up to the full compressive strength of the concrete, which condition will be practically true for corrugated bar reinforcement. The discussion further assumes that such a quantity of metal is used as will cause the elastic limit stress in the reinforcement and the full compressive strength of the concrete to be reached at the same time.

RECTANGULAR BEAMS

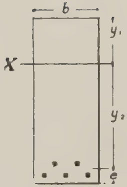


Fig. 1

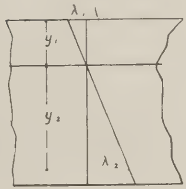


Fig. 2

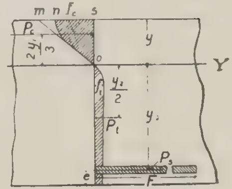


Fig. 3

Fig. 1 is a cross section of a reinforced concrete beam.

Fig. 2 represents the strain diagram at the ultimate load.

Fig. 3 is the stress diagram corresponding to the above strain diagram.

Let E_s=Modulus of elasticity of steel in pounds per square inch.

E_c=Modulus of elasticity of the concrete in compression in pounds per square inch.

F=Elastic limit of steel in pounds per square inch.

f_c=Compressive strength of concrete in pounds per square inch.

f_t=Tensile strength of concrete in pounds per square inch.
b=Width of section in inches.
a^2=Area of one bar in square inches.
d=Spacing of bars in inches.
$\frac{a^2}{d}$=Number of square inches of metal per inch of width.
$\frac{a^2b}{d}$=Total area of metal in width b.
M_o=Moment of ultimate resistance of cross-section in inch pounds.
M=Bending moment of external forces in inch pounds.
W=Total load on beam in pounds.
P_s=Total stress on metal in width b in pounds.
P_c=Total compressive stress on concrete in width b.
P_t=Total tensile stress in concrete in width b.
λ_1=Unit elongation of extreme fibre in compression.
λ_2=Unit elongation of steel.
c=Distance in inches from extreme fibre on tension side to middle plane of metal reinforcement. This thickness is not figured into the strength of the beam.

Referring to Fig. 3, we assume that the shaded area above the neutral axis represents the complete compressive stress diagram of the concrete, *o s* being the axis of proportionate elongation, and the neutral axis the axis of stress per square inch. From an examination of a great many such diagrams we have found that the resultant modulus—represented by the tangent of the angle *n o s*—is about two-thirds in rock concrete and one-half in cinder concrete as much as the original modulus—represented by the tangent of the angle *m o s*. Also that the total area for both kinds of concrete is about one-quarter larger than the triangular area *n o s*. These assumptions seem crude at first, but as a matter of fact they are not more so than would be any formula intended to represent the compressive stress diagram for a class of concrete. The latter would give all points on the curve, whereas our method gives only the end of same; but our location of that point is as accurate as can be obtained by any method.

We can then write the following equations:

ROCK CONCRETE

$$P_c = \tfrac{5}{8} f_c b y_1 \quad \cdots\cdots\cdots\cdots \quad (1)$$

$$f_c = \frac{2E_c\lambda_1}{3}$$

But $\lambda_1 = \lambda_2 \dfrac{y_1}{y_2}$

And $\lambda_2 = \dfrac{F}{E_s}$

Then $\lambda_1 = \dfrac{F y_1}{E_s y_2}$

And $f_c = \dfrac{2 F E_c y_1}{3 E_s y_2}$

Or $y_2 = \dfrac{2 F E_c}{3 f_c E_s} y_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$

For the steel,

$$P_s = \frac{F a^2 b}{d} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

For the concrete in tension,

$$P_t = \tfrac{8}{10} f_t b y_2 = \frac{8 F E_c f_t b y_1}{15 f_c E_s} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The empirical constant $\frac{8}{10}$ is derived from the results of M. Considère.

We then have,

$$P_c = P_s + P_t \dots\dots (5)$$

Or, $$\frac{5 f_c b y_1}{8} = \frac{F a^2 b}{d} + \frac{8 F E_c f_t b y_1}{15 f_c E_s} \dots\dots (6)$$

From which $$\frac{a^2 b}{d} = \frac{75 f_c b y_1 - 64 f_t b y_1 \left(\frac{F E_c}{f_c E_s}\right)}{120\ F} \dots\dots (7)$$

For the moment of resistance we have,

$$M_o = \frac{F a^2 b}{d}\left(y_2 + \frac{2 y_1}{3}\right) + \frac{8 f_t b y_2}{10}\left(\frac{y_2}{2} + \frac{2 y_1}{3}\right) \dots\dots (8)$$

The size of beam needed to develop a required moment of resistance can now be readily obtained from equations (2), (7) and (8). From (2) we obtain a numerical ratio between y_1 and y_2 when the constants depending only upon the particular materials used are known. Equation (7) gives the quantity of metal required in terms of y_1 all other factors being known constants for the given materials. Then (8) gives the value of the ultimate moment of resistance in terms of y_1 only. As the moment of resistance is to equal the bending moment of the external proof loads, M_o in equation (8) is known which at once gives the value of y_1 from which all other values may be determined.

AVERAGE ROCK CONCRETE

We have found the best average values for the constants for 1:3:6 rock concrete to be the following: $E_c=3{,}000{,}000$, $f_c=2{,}000$, and $f_t=200$.

For the steel the value of E_s varies but little for the different grades of rolled material, but F, or the elastic limit, varies greatly. As before stated in the introduction, we can not utilize any of the strength of the steel beyond the elastic limit, therefore it is desirable that this limit should be fairly high.

Our corrugated bars have an elastic limit of between 50,000 and 60,000 pounds per square inch. We therefore use for the constants for the steel, $E_s=29{,}000{,}000$ and $F=50{,}000$.

With these values equations (2), (7) and (8) reduce to the following respectively:

$$\left.\begin{aligned} y_2&=1.72y_1\\ \frac{a^2b}{d}&=.0195by_1\\ M_o&=2750by_1^2\\ h&=y_1+y_2+c \end{aligned}\right\} \begin{array}{l} \text{If } b=12''\\ \text{and } c=\dfrac{h}{10}\\ \text{we have,} \end{array} \left\{\begin{aligned} y_1&=.331h \;\dots\dots\dots\dots (9)\\ \frac{a^2b}{d}&=0.077h=.64\% \;\dots\dots (10)\\ M_o&=3620h^2 \;\dots\dots\dots\dots (11) \end{aligned}\right.$$

SPECIAL ROCK CONCRETE

There are certain grades of rock that give a much more compressible concrete than the above and have at the same time a greater com-

pressive strength. Trap rock falls within this category as well as certain kinds of western limestone using a well proportioned aggregate and a mix of 2:1:5. For such concrete we may assume the following constants:

$E_c = 2{,}400{,}000,\ f_c = 2{,}400,\ f_t = 200.$

Using the same values for the steel our equations of design then become:

$$\left.\begin{array}{l} y_2 = 1.15 y_1 \\ \dfrac{a^2 b}{d} = .0263 b y_1 \\ M_o = 2620 b y_1^2 \\ h = y_1 + y_2 + e \end{array}\right\} \begin{array}{l} \text{If } b = 12'' \\ \text{and } e = \dfrac{h}{10} \\ \text{we have,} \end{array} \left\{\begin{array}{ll} y_1 = .418h \ldots\ldots\ldots\ldots\ldots & (12) \\ \dfrac{a^2 b}{d} = 0.132h = 1.1\% \ldots\ldots\ldots & (13) \\ M_o = 5505 h^2 \ldots\ldots\ldots\ldots\ldots & (14) \end{array}\right.$$

CINDER CONCRETE

For a 1:2:5 mix of cinder concrete we have $E_c = 750{,}000$, $f_c = 750$ and $f_t = 80$.

For this material the equations become:

$$\left.\begin{array}{l} y_2 = 0.862 y_1 \\ \dfrac{a^2 b}{d} = .00827 b y_1 \\ M_o = 693 b y_1^2 \\ h = y_1 + y_2 + e \end{array}\right\} \begin{array}{l} \text{If } b = 12'' \\ \text{and } e = \dfrac{h}{10} \\ \text{we have,} \end{array} \left\{\begin{array}{ll} y_1 = .483h \ldots\ldots\ldots\ldots\ldots & (15) \\ \dfrac{a^2 b}{d} = 0.048h = .4\% \ldots\ldots\ldots & (16) \\ M_o = 1935 h^2 \ldots\ldots\ldots\ldots\ldots & (17) \end{array}\right.$$

TABLE FOR THE DESIGNING OF STEEL-CONCRETE BEAMS IN AVERAGE ROCK CONCRETE 1:3:6.

M	h	q	M	h	q
100	5.27	0.408	1000	16.68	1.289
150	6.45	.500	1500	20.40	1.580
200	7.45	.576	2000	23.50	1.812
250	8.32	.644	2500	26.30	2.038
300	9.12	.706	3000	28.80	2.230
350	9.85	.762	3500	31.15	2.410
400	10.52	.816	4000	33.25	2.578
450	11.15	.864	4500	35.25	2.730
500	11.73	.910	5000	37.20	2.880
550	12.38	.956	5500	39.10	3.025
600	12.90	.998	6000	40.80	3.160
650	13.40	1.040	6500	42.50	3.285
700	13.92	1.078	7000	44.00	3.410
750	14.40	1.113	7500	45.60	3.530
800	14.88	1.151	8000	47.00	3.640
850	15.34	1.188	8500	48.55	3.760
900	15.80	1.222	9000	49.90	3.860
950	16.25	1.258	10000	52.70	4.075

M=Ultimate bending moment of external forces in thousands of inch pounds.
h=Depth of beam in inches.
q=Number of square inches of metal required in beam one foot wide.
Depth to metal taken at $0.9h$.

TABLE FOR THE DESIGNING OF STEEL-CONCRETE BEAMS IN SPECIAL ROCK CONCRETE, 1:2:5.

M	h	q	M	h	q
100	4.27	0.562	1000	13.49	1.780
150	5.22	.689	1500	16.50	2.180
200	6.02	.795	2000	19.05	2.520
250	6.74	.889	2500	21.30	2.810
300	7.38	.975	3000	23.35	3.075
350	7.93	1.050	3500	25.20	3.325
400	8.52	1.125	4000	26.90	3.560
450	9.05	1.192	4500	28.59	3.780
500	9.53	1.258	5000	30.10	3.980
550	10.00	1.320	5500	31.60	4.180
600	10.44	1.380	6000	33.05	4.360
650	10.84	1.435	6500	34.39	4.530
700	11.29	1.486	7000	35.65	4.700
750	11.68	1.540	7500	36.90	4.870
800	12.02	1.588	8000	38.10	5.030
850	12.41	1.640	8500	39.30	5.190
900	12.79	1.686	9000	40.40	5.340
950	13.11	1.735	10000	42.60	5.620

M=Ultimate bending moment of external forces in thousands of inch pounds.
h=Depth of beam in inches.
q=Number of square inches of metal required in beam one foot wide.
Depth to metal taken at $0.9h$.

TABLE OF SPACING REQUIRED FOR DIFFERENT SIZES OF CORRUGATED BARS FOR GIVEN AREA OF METAL IN RECTANGULAR BEAMS ONE FOOT WIDE.

	OLD STYLE BAR					NEW STYLE BAR.						
C to C of Bar	½" BAR	¾" BAR	⅞" BAR	1" BAR	1¼" BAR	¼" BAR	½" BAR	⅝" BAR	¾" BAR	⅞" BAR	1" BAR	1¼" BAR
2"	1.08 "	2.22 "	3.30 "	4.20 "	6.43 "	0.36 "	1.50 "	2.34 "	3.36 "	4.62 "	6.00 "	9.37 "
2½"	0.86 "	1.78 "	2.65 "	3.36 "	5.14 "	0.29 "	1.20 "	1.87 "	2.69 "	3.70 "	4.80 "	7.50 "
3"	0.72 "	1.48 "	2.20 "	2.80 "	4.28 "	0.24 "	1.00 "	1.56 "	2.24 "	3.08 "	4.00 "	6.24 "
3½"	0.62 "	1.27 "	1.89 "	2.40 "	3.67 "	0.21 "	0.86 "	1.34 "	1.92 "	2.64 "	3.43 "	5.36 "
4"	0.54 "	1.11 "	1.65 "	2.10 "	3.21 "	0.18 "	0.75 "	1.17 "	1.68 "	2.31 "	3.00 "	4.68 "
4½"	0.48 "	0.99 "	1.47 "	1.86 "	2.85 "	0.16 "	0.67 "	1.04 "	1.49 "	2.05 "	2.67 "	4.16 "
5"	0.43 "	0.89 "	1.32 "	1.68 "	2.57 "	0.14 "	0.60 "	0.94 "	1.34 "	1.85 "	2.40 "	3.75 "
5½"	0.39 "	0.81 "	1.20 "	1.52 "	2.34 "	0.13 "	0.55 "	0.85 "	1.22 "	1.68 "	2.18 "	3.41 "
6"	0.36 "	0.74 "	1.10 "	1.40 "	2.14 "	0.12 "	0.50 "	0.78 "	1.11 "	1.53 "	2.00 "	3.12 "
6½"	0.33 "	0.68 "	1.02 "	1.29 "	1.97 "	0.11 "	0.46 "	0.72 "	1.03 "	1.42 "	1.85 "	2.88 "
7"	0.31 "	0.63 "	0.94 "	1.20 "	1.83 "	0.10 "	0.43 "	0.67 "	0.96 "	1.32 "	1.72 "	2.68 "
7½"	0.29 "	0.59 "	0.88 "	1.12 "	1.71 "	0.10 "	0.40 "	0.62 "	0.89 "	1.23 "	1.60 "	2.50 "
8"	0.27 "	0.55 "	0.82 "	1.05 "	1.60 "	0.09 "	0.38 "	0.59 "	0.81 "	1.15 "	1.50 "	2.34 "
8½"	0.25 "	0.52 "	0.77 "	0.99 "	1.51 "	0.08 "	0.35 "	0.55 "	0.79 "	1.09 "	1.42 "	2.20 "
9"	0.24 "	0.50 "	0.73 "	0.93 "	1.43 "	0.08 "	0.33 "	0.52 "	0.75 "	1.02 "	1.33 "	2.08 "
9½"	0.23 "	0.47 "	0.69 "	0.88 "	1.35 "	0.08 "	0.32 "	0.49 "	0.71 "	0.97 "	1.26 "	1.97 "
10"	0.22 "	0.44 "	0.66 "	0.84 "	1.28 "	0.07 "	0.30 "	0.47 "	0.67 "	0.92 "	1.20 "	1.87 "
11"	0.20 "	0.40 "	0.60 "	0.76 "	1.17 "	0.07 "	0.27 "	0.43 "	0.61 "	0.84 "	1.09 "	1.70 "
12"	0.18 "	0.37 "	0.55 "	0.70 "	1.07 "	0.06 "	0.25 "	0.39 "	0.56 "	0.77 "	1.00 "	1.56 "

TESTS OF THE UNION BETWEEN CONCRETE AND STEEL

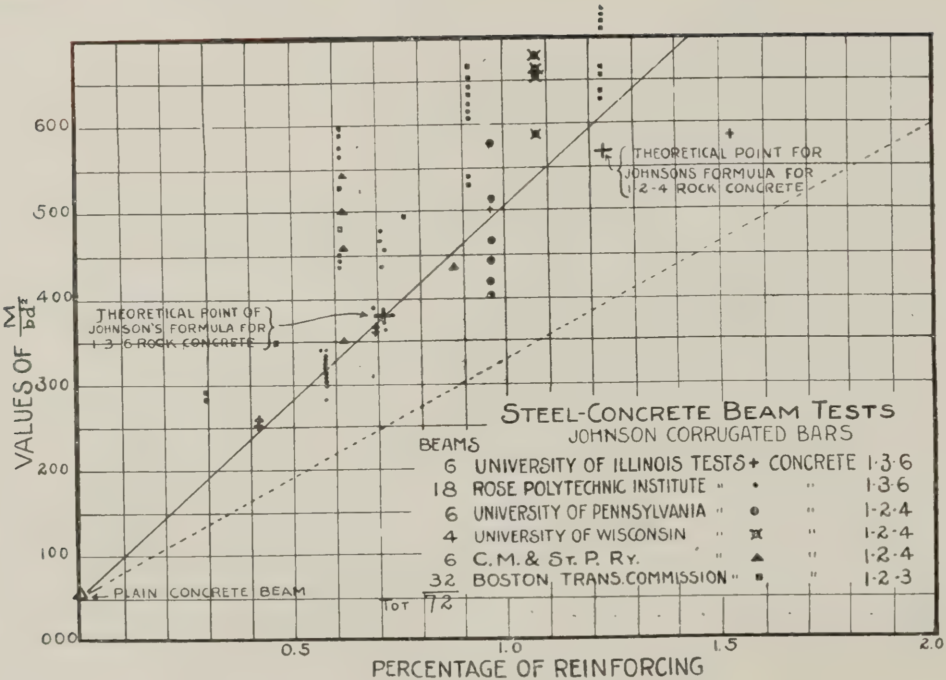
A recent issue of Beton and Eisen gave the results of a series of tests upon the holding power of different types of rods imbedded in concrete, made in the laboratories of the Massachusetts Institute of Technology by Prof. C. W. Spofford.

Portland cement concrete was used, made in the following proportions by weight: One part cement, three parts sand, six parts broken stone. This mixture was used in order that the results would correspond with tests upon beams and columns which were under way at the same time. The mixture, however, is very lean and would not again be used. The sand was clean, but rather coarse grained, containing approximately 47 per cent of voids. The broken stone was a mixture of two parts of 1″ trap and one part of ½″ trap. The mixing was thoroughly done by hand, the concrete being wet enough when tamped into the moulds to flush water to the surface. The moulds were, in some cases, not as tight as they should have been and some water leaked out, carrying with it some of the cement. It is not believed, however, that the loss thereby was sufficient to injure the results of the tests except possibly in a very few cases. The rods were all thoroughly cleaned by a sand blast, thus insuring uniformity in the surface conditions.

A 100,000-pound Olsen vertical testing machine was used, rigged with short uprights, carrying the platform upon which the specimens were placed. The load upon the bearing end of the concrete block was distributed by the interposition of a sheet of ½″ felt between the concrete and an angular steel ring resting upon the platform of the machine. In all cases the rod projected a short distance at the upper end of the block (the pull being downward at the lower end) and this projecting end was carefully watched in order to detect the first evidence of slipping. The rods used were round, square, flat, square but twisted through an angle of 20 degrees (Ransome rod), Thacher and Johnson. The table has been arranged from the original table in Beton and Eisen so that bars of the same size are together.—*Reprinted from the Railroad Gazette, for September 18, 1903.*

RESULTS OF TESTS ON THE UNION BETWEEN CONCRETE AND STEEL.

No. of test.	Type of rod, inch.		Size of concrete block, inch.	Length of rod imbedded in concrete, inch.	Breaking load, pounds.	Minimum area of cross-section of rod, square inch.	Shearing stress in pounds per square inch of net section.	Stress on rod in pounds per square inch of net section.	Remarks.
1	Ransome	1-2	6x6	12	12,100	0.25	504	48,400	Concrete split longitudinally
4	Ransome	1-2	8x8	12	8,300	0.25	346	33,200	Rod slipped at 8,000, dropped to 6,000, rose again to 8,300, where concrete split. Rod pulled through 3 in.
13	Thacher	1-2	6x6	12	4,850	0.18	270	26,900	Rod slipped, concrete split
22	**Johnson**	**1-2**	**6x6**	**12**	**12.200**	**0.14**	**678**	**87.200**	Concrete split
2	Ransome	1-2	6x6	16	8,100	0.25	253	32,400	Concrete split longitudinally
5	Ransome	1-2	8x8	16	14,000	0.25	438	56,000	Rod slipped at 12,000, dropped to 8 000, rose again to 14,000, where concrete split. Rod pulled through 5 in.
11	Thacher	1-2	6x6	16	8,200	0.18	340	45,500	Rod slipped at 8,100, concrete split
23	**Johnson**	**1-2**	**6x6**	**16**	**13.120**	**0.14**	**545**	**93.700**	Concrete split
3	Ransome	1-2	6x6	26	16,800	0.25	323	67,200	Concrete crushed on end
6	Ransome	1-2	8x8	26	15,000	0.25	288	60,000	Rod slipped at 15,000, rod pulled through max. stress 14,400. Rod pulled through 11 1-2 in.
15	Thacher	1-2	6x6	26	10,550	0.18	272	58,600	Rod broke
24	**Johnson**	**1-2**	**6x6**	**26**	**13.750**	**0.14**	**354**	**98.400**	Rod broke
7	Ransome	3-4	8x8	20	25,900	0.56	431	46,300	Rod slipped at 18,000, concrete split
16	Thacher	3-4	8x8	20	21,150	0.39	478	53,000	Rod slipped at 19,050, concrete split
25	**Johnson**	**3-4**	**8x8**	**20**	**27.600**	**0.31**	**619**	**89.100**	Concrete split
8	Ransome	3-4	8x8	24	31,900	0.56	443	57,000	Rod slipped at 14,000, concrete split
17	Thacher	3-4	8x8	24	18,300	0.39	344	45,900	Concrete split
26	**Johnson**	**3-4**	**8x8**	**24**	**25.000**	**0.31**	**467**	**80.600**	Concrete split
31	3-4 round		8x8	24	15,300	0.44	271	38,400	Rod slipped
34	3-4 square		8x8	24	19,700	0.56	274	35,200	Rod slipped
37	1 1-8 x 1-2		8x8	24	12,400	0.56	159	22,100	Rod slipped
40	1 1-2 x 3-8		8x8	24	20,300	0.56	226	36,300	Rod slipped
43	2 1-4 x 1-4		8x8	24	5,000	0.56	42	8,930	Rod slipped (specimen inj'd)
32	3-4 round		8x8	31	18,600	0.44	255	42,200	Rod slipped
35	3-4 square		8x8	31	22,600	0.56	243	40,400	Rod slipped
38	1 1-8 x 1-2		8x8	31	20,300	0.56	201	36,200	Rod slipped
41	1 1-2 x 3-8		8x8	31	21,700	0.56	188	38,800	Rod slipped
44	2 1-4 x 1-4		8x8	31	25,500	0.56	165	45,500	Rod slipped
9	Ransome	3-4	8x8	36	36,600	0.56	339	63,500	Concrete split
18	Thacher	3-4	8x8	36	23,700	0.39	297	59,400	Rod broke
27	**Johnson**	**3-4**	**8x8**	**36**	**28.000**	**0.31**	**483**	**90.500**	Concrete split
33	3-4 round		8x8	36	18,600	0.44	219	42,200	Rod slipped
36	3-4 square		8x8	36	23,900	0.56	221	42,700	Rod slipped
39	1 1-8 x 1-2		8x8	36	21,700	0.56	185	38,700	Rod slipped
42	1 1-2 x 3-8		8x8	36	22,130	0.56	164	39,500	Rod slipped
45	2 1-4 x 1-4		8x8	36	26,100	0.56	145	46,600	Rod slipped


STEEL-CONCRETE BEAM TESTS
JOHNSON CORRUGATED BARS
BEAMS
6 UNIVERSITY OF ILLINOIS TESTS + CONCRETE 1·3·6
18 ROSE POLYTECHNIC INSTITUTE " • " 1·3·6
6 UNIVERSITY OF PENNSYLVANIA " ● " 1·2·4
4 UNIVERSITY OF WISCONSIN " " 1·2·4
6 C.M. & ST. P. RY. " ▲ " 1·2·4
32 BOSTON TRANS. COMMISSION " ■ " 1·2·3
TOT 72
THEORETICAL POINT FOR JOHNSONS FORMULA FOR 1-2-4 ROCK CONCRETE
THEORETICAL POINT OF JOHNSON'S FORMULA FOR 1-3-6 ROCK CONCRETE
PLAIN CONCRETE BEAM
VALUES OF $\frac{M}{bd^2}$
PERCENTAGE OF REINFORCING
000
100
200
300
400
500
600
0.5
1.0
1.5
2.0

FLOOR PANELS

The foregoing discussion applies to beams on knife edge supports. Rectangular beams when incorporated in floor panels will have just about twice the capacity given by the formula, and the following tables, I to VI, are made up on this basis.

To give a scientific discussion of this is almost impossible. It is a matter of actual practical experience. We can, however, see that it is reasonable to expect about such an increase. The haunches built down upon the lower flange of the supporting beams give a continuous girder action such as reduces the external bending moment one-third. Also the floor in adjacent panels produces an interior arching action, increasing the area of this compressive stress diagram about one-third, the effect of the two being to double the moment of resistance.

If the beam does not have the haunches projecting below as described, but is itself the full depth throughout, then we would add one-third only to the value of the moment of resistance.

Beams of Tee shape are not greatly strengthened by incorporation in floor panels inasmuch as most of the compressive strength comes from the flanges, too high up to be affected by the interior arching action. That is to say, P_c'' (see page 135) would remain practically the same and P_c' would be increased probably 50 per cent. But the latter is usually so small as to make this increase of little value.

TABLE I.

GIVING BREAKING LOADS FOR CINDER CONCRETE FLOOR SLABS WITH No. 16GA. $2\frac{1}{2}''$ MESH EXPANDED METAL IMBEDDED.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	SPAN IN FEET.														$M_\circ''$=Floor-Slab Moment of Resistance $=2 M_\circ$
	4		5		6		7		8		9		10		
	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
2	680	0 68	435	0.54	300	0 45	....	...	...	....	....	....	....	...	16300
$2\frac{1}{2}$	1060	1.06	680	0.85	470	0 77	345	0.61	...	....	....	....	....	....	25460
3	1360	1 36	870	1.09	605	0 91	445	0.78	340	0.68	....	...	....	...	32830
$3\frac{1}{2}$	1640	1.64	1050	1.31	725	1.09	535	0.94	410	0.82	325	0.73	....	....	39240
4	1900	1 90	1220	1.52	845	1.27	620	1 09	475	0.95	380	0.85	305	0.76	45700
$4\frac{1}{2}$	2180	2 18	1390	1.74	970	1.45	710	1 24	545	1.09	430	0.97	350	0.87	52200
5	2450	2.45	1560	1 96	1090	1 63	795	1.40	610	1 22	485	1.09	390	0.98	58750
$5\frac{1}{2}$	2740	2.74	1740	2.17	1210	1 81	890	1 55	680	1 36	540	1.21	440	1 09	65300
6	3000	3 00	1910	2.39	1330	1.99	975	1.71	750	1.49	590	1.33	480	1.20	71900

$U=\frac{M_\circ''}{1.5\, l^2}$ $C=\frac{M_\circ''}{6000\, l}$ l=span in feet.

TABLE II.

GIVING BREAKING LOADS FOR CINDER CONCRETE FLOOR SLABS WITH No. 10GA. 3″ MESH EXPANDED METAL IMBEDDED.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	SPAN IN FEET.														M_0''=Floor-Slab Moment of Resistance $=2M_0$
	4		5		6		7		8		9		10		
	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
2	720	0.72	460	0.58	320	0.48	....	....	....	....	....	....	....	....	17350
2½	1130	1.13	730	0.91	505	0.76	370	0.65	....	....	....	....	....	....	27200
3	1620	1.62	1035	1.29	720	1.08	525	0.92	405	0.81	....	....	....	....	38800
3½	2140	2.14	1370	1.71	950	1.42	700	1.22	535	1.07	425	0.95	....	....	51300
4	2490	2.49	1595	1.99	1110	1.66	815	1.42	620	1.24	490	1.11	400	1.00	59800
4½	2860	2.86	1820	2.28	1270	1.90	930	1.62	710	1.42	565	1.26	455	1.14	68300
5	3200	3.20	2050	2.56	1430	2.13	1050	1.83	800	1.60	630	1.42	510	1.28	76900
5½	3560	3.56	2280	2.85	1580	2.37	1165	2.03	890	1.78	705	1.58	570	1.42	85500
6	3950	3.95	2520	3.14	1750	2.62	1280	2.24	980	1.96	775	1.74	630	1.57	94200

$U=\frac{M_0''}{1.5\,l^2}$ $C=\frac{M_0''}{6000\,l}$ l=span in feet.

TABLE III.

GIVING BREAKING LOADS FOR CINDER CONCRETE FLOOR SLABS, USING ½″ SQUARE CORRUGATED STEEL BARS OF SUCH SPACING AS TO MAKE THE SLABS OF EQUAL STRENGTH IN TENSION AND COMPRESSION.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	Spacing of Bars in inches.	SPAN IN FEET. 8		9		10		11		12		13		14		15		16		Mo″ Floor-Slab Moment of Resistance =2[Mo or Mo′]
		U	C	U	C	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
3½	13	390	0.78	310	0.69	...	...	...	...	...	...	...	...	...	...	...	...	...	...	37500
4	11	550	1.09	430	0.97	350	0.87	...	...	...	...	...	...	...	...	...	...	...	...	52400
4½	9½	730	1.46	575	1.30	465	1.17	385	1.06	...	...	...	...	...	...	...	...	...	...	70000
5	8½	930	1.85	730	1.65	590	1.48	490	1.35	410	1.24	...	...	...	...	...	...	...	...	89000
5½	7½	1170	2.34	930	2.08	750	1.87	620	1.70	520	1.56	445	1.44	385	1.34	...	...	...	...	112400
6	7	1390	2.77	1090	2.46	885	2.21	735	2.02	615	1.84	525	1.71	455	1.58	395	1.48	...	...	133000
6½	6	1770	3.54	1400	3.15	1130	2.83	935	2.57	790	2.36	670	2.18	580	2.02	505	1.89	440	1.76	170000
7	5½	2100	4.21	1660	3.74	1350	3.37	1110	3.06	935	2.81	800	2.59	685	2.41	600	2.25	525	2.10	202000
7½	5	2500	5.00	1970	4.45	1600	4.00	1320	3.64	1110	3.34	945	3.08	815	2.86	710	2.67	625	2.50	240000

$U = \frac{M_o''}{1.5\, l^2}$ $C = \frac{M_o''}{6000\, l}$ l=span in feet.

TABLE IV.

GIVING BREAKING LOADS FOR ROCK CONCRETE FLOOR SLABS WITH No. 16GA. $2\frac{1}{2}''$ MESH EXPANDED METAL IMBEDDED.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	SPAN IN FEET.														$M_\circ''$=Floor-Slab Moment of Resistance $=2 M_\circ$
	4		5		6		7		8		9		10		
	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
2	930	0.93	595	0.75	415	0 62	...	...	....	...	...	....	...	....	22450
$2\frac{1}{2}$	1210	1.21	780	0 97	540	0.81	400	0.69	...	...	...	...	....	....	29200
3	1500	1.50	960	1.20	665	1.00	490	0.86	375	0.75	....	...	....	...	36000
$3\frac{1}{2}$	1780	1 78	1140	1.43	790	1.19	580	1.02	445	0.89	350	0.79	....	.. .	42850
4	2070	2.07	1330	1.66	920	1 38	675	1.18	520	1.03	410	0.92	330	0.83	49700
$4\frac{1}{2}$	2360	2.36	1510	1.89	1050	1.57	770	1.35	590	1.18	465	1 05	375	0.94	56600
5	2650	2.64	1690	2.12	1180	1.76	865	1.51	660	1.32	520	1.18	425	1.06	63500
$5\frac{1}{2}$	2930	2.93	1880	2.35	1300	1.96	960	1.67	735	1.47	580	1.30	470	1.17	70400
6	3220	3.22	2060	2.57	1430	2 15	1050	1.84	810	1.61	640	1.43	520	1.29	77300

$U=\frac{M_\circ''}{1.5\,l^2}$ $C=\frac{M_\circ''}{6000\,l}$ l=span in feet.

TABLE V.

GIVING BREAKING LOADS FOR ROCK CONCRETE FLOOR SLABS WITH No. 10GA. 3″ MESH EXPANDED METAL IMBEDDED.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	SPAN IN FEET.														M_o''=Floor-Slab Moment of Resistance $=2 M_o$
	4		5		6		7		8		9		10		
	U	C	U	C	U	C	U	C	U	C	U	C	U	C	
2	1230	1.23	785	0.98	545	0.82	400	0.70	...	...	....	....	....	....	29500
$2\frac{1}{2}$	1600	1.60	1020	1.28	710	1.06	520	0.91	400	0.80	....	....	....	....	38400
3	1970	1.97	1260	1.58	875	1.32	645	1.13	495	0.99	390	0.88	....	....	47400
$3\frac{1}{2}$	2350	2.35	1500	1.88	1050	1.57	770	1.34	590	1.17	465	1.04	375	0.94	56450
4	2730	2.73	1750	2.18	1210	1.82	890	1.56	680	1.36	540	1.21	435	1.09	65500
$4\frac{1}{2}$	3110	3.11	1990	2.49	1380	2.07	1010	1.78	775	1.55	615	1.38	495	1.24	74700
5	3490	3.49	2230	2.79	1550	2.33	1140	1.99	875	1.74	690	1.55	560	1.39	83850
$5\frac{1}{2}$	3870	3.87	2480	3.10	1720	2.58	1265	2.21	970	1.94	765	1.72	620	1.55	93000
6	4260	4.26	2740	3.41	1900	2.84	1400	2.44	1070	2.14	840	1.90	680	1.71	102200

$U=\frac{M_o''}{1.5\,l^2}$ $C=\frac{M_o''}{6000\,l}$ l=span in feet.

TABLE VI.

GIVING BREAKING LOADS FOR ROCK CONCRETE FLOOR SLABS, USING ½″ SQUARE CORRUGATED STEEL BARS OF SUCH SPACING AS TO MAKE THE SLABS OF EQUAL STRENGTH IN TENSION AND COMPRESSION.

U=Uniformly distributed load in pounds per square foot, in addition to dead weight.
C=Concentrated load in tons, in middle of slab 12″ wide.

Thickness of Slab in inches.	Spacing of Bars in inches.	SPAN IN FEET. 8 U	8 C	9 U	9 C	10 U	10 C	11 U	11 C	12 U	12 C	13 U	13 C	14 U	14 C	15 U	15 C	16 U	16 C	M_o''=Floor-Slab Moment of Resistance =2[M_o or M_o']
3½	7	775	1.55	610	1.38	495	1.24	410	1.13	....	....	....	....	....	....	....	....	....	....	74400
4	6	1070	2.14	840	1.90	685	1.71	565	1.56	475	1.43	405	1.32	....	....	....	....	....	....	102700
4½	5	1480	2.96	1165	2.63	945	2.36	780	2.15	660	1.97	560	1.82	480	1.69	420	1.58	....	....	142000
5	4½	1860	3.73	1470	3.31	1190	2.98	985	2.71	830	2.48	705	2.29	610	2.13	530	1.99	465	1.86	179000
5½	4	2340	4.68	1850	4.16	1500	3.75	1240	3.40	1040	3.12	885	2.88	765	2.68	665	2.50	585	2.35	225000
6	3½	2950	5.90	2330	5.25	1890	4.74	1560	4.30	1310	3.94	1120	3.65	965	3.38	840	3.15	740	2.96	284000
6½	3½	3250	6.50	2560	5.78	2080	5.20	1720	4.72	1440	4.34	1230	4.00	1060	3.71	920	3.46	810	3.24	311000
7	3	4100	8.24	3250	7.30	2630	6.58	2170	5.98	1830	5.48	1560	5.05	1340	4.70	1170	4.39	1030	4.12	395000
7½	3	4450	8.88	3500	7.88	2850	7.10	2350	6.45	1980	5.92	1680	5.46	1450	5.08	1260	4.75	1110	4.44	426000

$U=\frac{M_o''}{1.5\,l^2}$ $C=\frac{M_o''}{6000\,l}$ l=span in feet.

TABLE FOR DESIGNING HIGHWAY CULVERT COVERS

W	Span	3′		4′		5′		6′		7′		8′		9′		10′		11′	
	Fill	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$
1800	1′	4.3	.20	5.1	.27	5.9	.34	6.7	.40	7.4	.47	8.2	.54	9.0	.60	9.8	.67	10.6	.73
2100	2′	4.5	.22	5.4	.29	6.2	.36	7.0	.43	7.8	.52	8.7	.58	9.5	.64	10.4	.72	11.2	.79
2400	3′	4.7	.24	5.6	.31	6.5	.39	7.4	.46	8.3	.55	9.2	.62	10.1	.68	11.0	.77	11.8	.83
2700	4′	4.9	.25	5.8	.33	6.7	.42	7.7	.49	8.6	.57	9.7	.65	10.6	.73	11.5	.82	12.4	.89
3000	5′	5.0	.26	6.0	.35	7.0	.44	8.0	.52	9.0	.60	10.1	.69	11.1	.78	12.0	.87	13.0	.96
3300	6′	5.2	.27	6.2	.36	7.3	.46	8.3	.54	9.4	.63	10.4	.72	11.5	.81	12.5	.90	13.5	1.00
3600	7′	5.3	.28	6.4	.38	7.5	.48	8.6	.56	9.7	.66	10.6	.75	11.9	.84	13.0	.94	14.1	1.04
3900	8′	5.4	.29	6.5	.40	7.7	.50	8.9	.58	10.0	.69	11.2	.78	12.3	.88	13.5	.97	14.6	1.08
4200	9′	5.6	.30	6.7	.41	7.9	.52	9.2	.61	10.3	.72	11.6	.82	12.8	.91	14.0	1.01	15.1	1.12
4500	10′	5.7	.32	6.9	.42	8.2	.53	9.4	.63	10.6	.74	11.9	.85	13.2	.95	14.4	1.05	15.6	1.15
4800	11′	5.8	.33	7.1	.44	8.4	.54	9.6	.65	10.9	.76	12.2	.88	13.5	.98	14.8	1.08	16.0	1.19
5100	12′	5.9	.34	7.3	.45	8.6	.56	9.9	.67	11.2	.78	12.5	.90	13.8	1.01	15.1	1.12	16.4	1.24
5400	13′	6.0	.35	7.4	.47	8.8	.57	10.1	.69	11.5	.80	12.8	.93	14.1	1.04	15.5	1.15	16.8	1.28
5700	14′	6.1	.36	7.5	.48	9.0	.59	10.4	.72	11.7	.83	13.1	.95	14.5	1.07	15.9	1.19	17.3	1.32
6000	15′	6.3	.37	7.7	.49	9.1	.61	10.6	.74	11.9	.85	13.4	.98	14.9	1.10	16.4	1.23	17.8	1.36

T=Thickness concrete roof in inches.

$\frac{a^2b}{d}$=Area (in □″) of steel required per foot width.

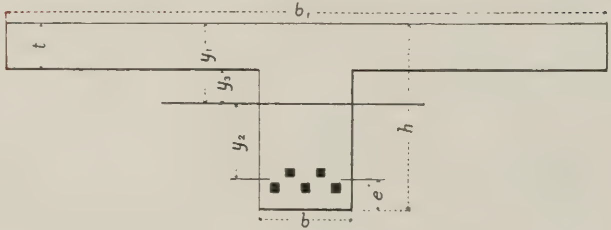
IN REINFORCED CONCRETE CONSTRUCTION WITH CORRUGATED BARS.

W	Span	12′		13′		14′		15′		16′		17′		18′		19′		20′	
	Fill	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$	T	$\frac{a^2b}{d}$
1800	1′	11.4	.81	12.1	.86	12.9	.93	13.7	.99	14.5	1.06	15.3	1.13	16.0	1.20	16.8	1.27	17 6	1.34
2100	2′	12.0	.86	12 8	.92	13.7	.99	14.5	1.06	15.3	1 14	16 2	1.21	17 0	1.29	17 9	1.36	18 8	1.45
2400	3′	12.7	.92	13 6	.98	14.5	1.06	15.3	1.13	16 2	1.22	17.1	1 29	18 0	1.37	18 9	1.45	20 0	1 54
2700	4′	13.3	.98	14.4	1 05	15.3	1.13	16.1	1.21	17 2	1.29	18.1	1.37	19.0	1.46	20.0	1.54	21.1	1.64
3000	5′	14.0	1 04	15.1	1 12	16.1	1.20	17 0	1.28	18 1	1.32	19 1	1.46	20.1	1.55	21.1	1.64	22 1	1.75
3300	6′	14.6	1.09	15.7	1 17	16.7	1.25	17.7	1.35	18.9	1.43	19 9	1 52	21.0	1.62	22.0	1.71	23 0	1 82
3600	7′	15.2	1.13	16 3	1.21	17.4	1.30	18.5	1.41	19.6	1.49	20.7	1.58	21 8	1.69	22 9	1.78	24 0	1 89
3900	8′	15.7	1.17	17.0	1 27	18.0	1.36	19 2	1 47	20 3	1.55	21.4	1.64	22.5	1.76	23.7	1.85	24.8	1 97
4200	9′	16.3	1 22	17.6	1 32	18.6	1.41	19.8	1.53	21.0	1 61	22 1	1.72	23.3	1.83	24.5	1.95	25.7	2.04
4500	10′	16.9	1.26	18 1	1.36	19.3	1 47	20 5	1.58	21.7	1.68	22.9	1 78	24 1	1.90	25.3	2 01	26.6	2.12
4800	11′	17.4	1.30	18.5	1 41	19.8	1.52	21 1	1.63	22.3	1.73	23.6	1.84	24.8	1.96	26 0	2.07	27.4	2.19
5100	12′	17 8	1.34	19.0	1 46	20.3	1 57	21 7	1.68	23 0	1.78	24.3	1.90	25.5	2.02	26.8	2.13	28.1	2.25
5400	13′	18.2	1.38	19.5	1.51	20.9	1 62	22.2	1.73	23 6	1.84	25.0	1 95	26.3	2 08	27.6	2.19	28 9	2.31
5700	14′	18.7	1 42	20.1	1 56	21.6	1.67	23.0	1.78	24.5	1.90	25.9	2 02	27.3	2.14	28 7	2 25	29.8	2.38
6000	15′	19 3	1 48	20.8	1.60	22.3	1.72	23 9	1.85	25.4	1.96	26.9	2.07	28.4	2.20	29.9	2 32	30.4	2.44

W=Uniformly distributed breaking load in pounds per square foot (includes road roller, 24 tons, on 120□′).

Note.—Factor of safety: 4 on live load, 2 on dead load.

TEE-SHAPED BEAMS



LOCATION OF NEUTRAL AXIS

In beams of Tee section y_1 is the same as for rectangular sections inasmuch as the position of the neutral axis is determined by the relative values of maximum compressibility of the concrete and extensibility of the steel inside the elastic limit or by the ratio of λ_1 and λ_2. This is of course only true at the maximum load.

We then have as before,

$$y_2 = \frac{2FE_c}{3f_cE_s}y_1 \text{ .... } (18)$$

VALUES OF b_1 AND t.

Let S_v = Total shear in pounds along the two vertical planes of attachment between the wings and beam;

S_h = Total shear in pounds along the horizontal plane of attachment between the rib and floor plate;

s = Maximum shearing strength of concrete in pounds per square inch;

$K = \frac{y_3}{y_1}$

l = Length of span in feet;

P_c' = Total compression in pounds at maximum load between neutral axis and underside of floor plate;

P_c'' = Total compression in pounds in flange at maximum load.

All other functions as shown on cut, and in inches.

There are three methods of failure above the neutral axis:

1. By compression in the flange;
2. By deficiency in S_v owing to smallness of t;
3. By deficiency in S_h owing to smallness of b.

It would be desirable to have equal strength in all these directions, but this is not always possible owing to other considerations. Where it is possible we have,

$$P_c''=S_v=S_h \quad \text{.......... (19)}$$

But $S_h=3bsl$ .... (20)

and $S_v=6tsl$ .... (21)

The shearing stress is a maximum at the ends and for uniformly loaded beam varies uniformly to zero at the center. The value S_v may be increased about 50 per cent owing to the metal reinforcement in the underside of floor plate which is always present in these designs. If vertical shear bars were used the same increase could be made in S_h, but ordinarily these would not be used so we will not separately discuss this condition. Equation (21) then becomes

$$S_v=9tsl \quad \text{.......... (22)}$$

Assuming the compression stress diagram to be a parabola

$$P_c''=\tfrac{2}{3}\,(1-K^{3/2})\,f_c\,b_1\,y_1 \quad \text{.......... (23)}$$

This is on the assumption that the outer ends of the wings would be just as heavily stressed as the portion next to the beam. This would not be the case, the stress varying according to the ordinates to a parabola from zero at the outer ends to a maximum at the beam, and

we should, therefore, multiply the above value by $\frac{2}{3}$. The portion of this width over the beam itself would not be subject to this modification, but there are other influences tending to offset this so that the above is sufficiently correct.

Then $P_c'' = \frac{4}{9}\,(1 - K^{3/2})\, f_c b_1 y_1$ .... (24)

From (20) and (22) we see that if t is not less than $\frac{b}{3}$, failure will not occur along the vertical sides of beam where wings attach. Now we will assume at once that t will not be allowed to have a value less than this. This leaves us to consider the relation between P_c'' and S_h only. We then have from (20) and (24)

$$3bsl = \frac{4}{9}(1 - K^{3/2})\, f_c b_1 y_1 \text{ from which}$$

$$b_1 = \frac{27bsl}{4(1 - K^{3/2})\, f_c y_1} \qquad \text{(25)}$$

The theoretical relation between s and f_c is

$$s = \frac{f_c}{2\tan\theta} \text{ (see Johnson's Materials of Construction, p. 29)} \qquad \text{(26)}$$

where θ is the angle made by the plane of rupture on a compression specimen of moderate length with a plane at right angles to the direction of stress.

For concrete this angle is about 60°, hence

$$s=\frac{f_c}{3.464} \qquad (27)$$

But this value is high in view of the liability of concrete to crack and we recommend that twice the strength be provided in the shearing values on this basis that is used in compression.

We would then have $S_h=2P_c''$ or

$3bsl=\frac{8}{9}(1-K^{3/2})\,f_c b_1 y_1$ from which

$b_1=\frac{27bsl}{8(1-K^{3/2})\,f_c y_1}$ and substituting the value of s

we have with sufficient accuracy,

$$b_1=\frac{bl}{(1-K^{3/2})\,y_1} \qquad (28)$$

We will now insert this value in (24) and proceed to obtain the moment of resistance. At times the above value of b_1 would be greater than the spacing of the beams, in which case the latter distance would be used for the value of b_1 in (24) and the other values worked over on this basis.

From (24) and (28) then we have,

$$P_c'' = \frac{4}{9} f_c b l \quad \text{.......... (29)}$$

also $$P_c' = \tfrac{2}{3} K^{3/2} f_c b y_1 \quad \text{.......... (30)}$$

Then $$P_c = P_c' + P_c'' = \frac{2 f_c b}{3}\left(\frac{2l}{3} + K^{3/2} y_1\right) \quad \text{.......... (31)}$$

$$P_t = .8 f_t b y_2 \quad \text{.......... (32)}$$

$$P_s = \frac{F a^2 b}{d} \quad \text{.......... (33)}$$

But $$P_c = P_t + P_s \quad \text{.......... (34)}$$

From which

$$\frac{a^2 b}{d} = \frac{1}{F}\left[\frac{2 f_c b}{3}\left(\frac{2l}{3} + K^{3/2} y_1\right) - .8 f_t b y_2\right] \quad \text{.......... (35)}$$

and $$M_o = P_c' \frac{K y_1}{2} + P_c'' \frac{(1+K) y_1}{2} + P_t \frac{y_2}{2} + P_s y_2 \quad \text{.......... (36)}$$

Problem: Required the size of Tee-shaped beam necessary to carry a total ultimate load of 600 pounds per square foot on a span of 32 feet, ribs to be 9 feet apart.

Then $$M=\frac{12\text{x}9\text{x}600\text{x}1024}{8}=8{,}300{,}000 \text{ inch pounds.}$$

Let us assume a depth of beam h equal to 22″. Then $y_1+y_2=20''$. For this spacing of beams the thickness of floor plate should be 4″.

Using special rock concrete we have from (12)

$$y_1=\frac{20}{2.15}=9.3'' \text{ and } y_2=10.7''$$

$$K=\frac{y_1-t}{y_1}=\frac{5.3}{9.3}=.57 \text{ and } K^{3/2}=.43$$

Then

$$P_c'=\tfrac{2}{3}K^{3/2}f_c b y_1=\tfrac{2}{3}\times .43\times 2400\times 9.3b= 6400b$$

$$P_c''=\frac{4}{9}f_c b l \qquad =\frac{4}{9}\times 2400\times 32b \qquad =34100b$$

and $$P_c \qquad =40500b$$

$$P_t=.8f_t b y_2=.8\times 200\times 10.7b \qquad = 1715b$$

Then $$P_s \qquad =38785b$$

and

$$\frac{a^2b}{d}=\frac{38785b}{50000}=.776b$$

$$M_o = P_c' \frac{K y_1}{2} + P_c'' \frac{(1+K) y_1}{2} + P_t \frac{y_2}{2} + P_s y_2$$

$$= 6400b \times 2.65 + 34100b \times 7.3 + 1715b \times 5.35 + 38785b \times 10.7$$

$$= 690130b$$

or, $$b = \frac{8,300,000}{690130} = 12.03''$$

Substituting in (28) we have

$$b_1 = \frac{12 \times 32}{.57 \times 9.3} = 72.5'' \text{ or } 6'.$$

As this value of b_1, which we have used in determining the value of P_c'' above, is less than the spacing of the beams it is the proper one to have used. It will be noted that t is just one-third of b.

From the foregoing we derive the following relations for good grade of 1:2:5 Portland cement rock, concrete, where $f_c = 2400$; $f_t = 200$; $E_c = 2,400,000$; $E_s = 29,000,000$; $F = 50,000$.

$$P_c' = 1600 K^{3/2} b y_1; \; P_t = 160 b y_2; \; P_c'' = 1066 b l.$$

Also $\frac{a^2 b}{d} = \frac{P_c' - P_t + P_c''}{50,000}$ = number of square inches of metal required in rib.

$$M_o = P_c'\left(\frac{y_1}{2} + y_2 - \frac{t}{2}\right) + P_c''\left(y_1 + y_2 - \frac{t}{2}\right) - P_t\frac{y_2}{2}$$ =ultimate moment of resistance in inch pounds.

All measures of length in inches except l, the length of span, which is in feet.

The value of t must not be less than one-third b.

The value of b_1 represents the maximum width of flange that can be utilized in figuring the strength of the Tee, and its value is:

$b_1 = \dfrac{bl}{(1-K^{3/2})y_1}$. Where this value of b_1 exceeds materially the distance between the ribs, the above formulæ and the following table could not be used, and the value of P_c'' would have to be obtained from the general equation (24).

The values in the following table are based upon the foregoing values for good rock concrete:

TABLE FOR THE DESIGN OF TEE BEAMS.

t	h	e	y_1	y_2	K	$K^{3/2}$	Area of Steel	Ultimate moment	Panel width b_1
3	9	2.0	3.26	3.74	.080	.023	b(—.0096 + .0213 l)	b(— 654 + 5866 l)	.314 bl.
	10	2.0	3.72	4.28	.193	.085	b(—.0036 + .0213 l)	b(+ 882 + 6932 l)	.294 bl.
	11	2.0	4.2	4.8	.286	.153	b(+.0052 + .0213 l)	b(3708 + 8000 l)	.281 bl.
	12	2.0	4.65	5.35	.355	.212	b(+.0144 + .0213 l)	b(7448 + 9067 l)	.272 bl.
	13	2.2	5.0	5.8	.400	.253	b(+.0219 + .0213 l)	b(11072 + 9920 l)	.268 bl.
	14	2.4	5.4	6.2	.445	.297	b(+.0315 + .0213 l)	b(15917 + 10773 l)	.263 bl.
	15	2.5	5.8	6.7	.483	.336	b(+.0409 + .0213 l)	b(21667 + 11734 l)	.260 bl.
	16	2.7	6.2	7.1	.516	.371	b(+.0509 + .0213 l)	b(27983 + 12587 l)	.256 bl.
	17	2.8	6.6	7.6	.545	.400	b(+.0602 + .0213 l)	b(35085 + 13547 l)	.253 bl.
	18	3.0	7.0	8.0	.570	.430	b(+.0707 + .0213 l)	b(43040 + 14400 l)	.250 bl.
	19	3.0	7.4	8.6	.596	.460	b(+.0814 + .0213 l)	b(53322 + 15474 l)	.250 bl.
	20	3.0	7.9	9.1	.620	.488	b(+.0942 + .0213 l)	b(64609 + 16533 l)	.247 bl.
	21	3.0	8.4	9.6	.642	.515	b(+.1077 + .0213 l)	b(77768 + 17596 l)	.245 bl.
	22	3.0	8.8	10.2	.660	.536	b(+.1183 + .0213 l)	b(90572 + 18667 l)	.245 bl.
4	11	2.0	4.2	4.8	.047	.010	b(—.0140 + .0213 l)	b(—1514 + 7467 l)	.240 bl.
	12	2.0	4.65	5.35	.140	.052	b(—.0094 + .0213 l)	b(— 93 + 8533 l)	.227 bl.
	13.2	2.2	5.12	5.88	.218	.102	b(—.0021 + .0213 l)	b(+ 2616 + 9600 l)	.217 bl.
	14.4	2.4	5.6	6.4	.286	.153	b(+.0069 + .0213 l)	b(6595 + 10667 l)	.211 bl.
	15.5	2.5	6.05	6.95	.339	.197	b(+.0158 + .0213 l)	b(11320 + 11733 l)	.206 bl.
	16.6	2.6	6.5	7.5	.385	.239	b(+.0257 + .0213 l)	b(17252 + 12800 l)	.202 bl.
	17.8	2.8	7.0	8.0	.429	.281	b(+.0373 + .0213 l)	b(24777 + 13867 l)	.199 bl.
	19.2	3.2	7.4	8.6	.459	.311	b(+.0461 + .0213 l)	b(32008 + 14933 l)	.196 bl.
	20.4	3.4	7.9	9.1	.493	.346	b(+.0583 + .0213 l)	b(41697 + 16000 l)	.194 bl.
	21.5	3.5	8.4	9.6	.524	.379	b(+.0712 + .0213 l)	b(52736 + 17067 l)	.192 bl.
	22.6	3.6	8.8	10.2	.546	.403	b(+.0808 + .0213 l)	b(63169 + 18133 l)	.190 bl.
5	14.3	2.3	5.6	6.4	.107	.035	b(—.0142 + .0213 l)	b(—1176 + 10133 l)	.185 bl.
	15.6	2.6	6.05	6.95	.174	.073	b(—.0081 + .0213 l)	b(+ 1421 + 11200 l)	.178 bl.
	17.0	3.0	6.5	7.5	.243	.120	b(+.0010 + .0213 l)	b(5796 + 12267 l)	.175 bl.
	18.0	3.0	7.0	8.0	.286	.153	b(+.0087 + .0213 l)	b(10306 + 13333 l)	.169 bl.
	19.2	3.2	7.4	8.6	.325	.185	b(+.0163 + .0213 l)	b(15545 + 14400 l)	.166 bl.
	20.4	3.4	7.9	9.1	.367	.222	b(+.0270 + .0213 l)	b(22978 + 15467 l)	.163 bl.
	21.5	3.5	8.4	9.6	.404	.257	b(+.0384 + .0213 l)	b(31657 + 16533 l)	.160 bl.
	22.7	3.7	8.8	10.2	.432	.284	b(+.0473 + .0213 l)	b(40064 + 17600 l)	.159 bl.
	24.0	4.0	9.3	10.7	.463	.315	b(+.0595 + .0213 l)	b(51069 + 18667 l)	.157 bl.
	25.0	4.0	9.77	11.23	.489	.342	b(+.0710 + .0213 l)	b(62696 + 19733 l)	.155 bl.

SHEAR IN REINFORCED CONCRETE BEAMS

Let M_1 = moment of resistance in inch pounds at 12″ from end of beam carrying its ultimate load.

M_0 = ultimate moment of resistance in inch pounds at center.

l = span of beam in feet.

λ_2 = elongation per inch at the plane of the metal, at section 12″ from end.

b = width of beam in inches.

s = ultimate shearing strength of the concrete, about one-fourth the ultimate compressive strength.

Other functions as shown on pages 113 and 114.

Then $M_1 = \dfrac{4l-4}{l^2} M_0$ for uniformly loaded beam .... (1)

$$\lambda_2 = \frac{M_1}{\dfrac{E_c\, b y_1{}^3}{3y_2} + \dfrac{E_c\, b y_2{}^2}{3} + \dfrac{E_s\, a^2 b y_2}{d}} \qquad (2)$$

$$b y_1{}^2 = b y_2{}^2 + \frac{2\, E_s\, a^2 b}{E_c\, d} y_2 \qquad (3)$$

$$y_1 = h - y_2 - e \qquad (4)$$

$$P_s = \frac{E_s\, \lambda_2\, a^2 b}{d} \qquad (5)$$

After designing the beam by the beam formulæ, pages (118) and (119) $\dfrac{a^2 b}{d}$ $y_1 + y_2$, E_c, E_s, and b are known. From (1) we obtain M_1 and from (3) and (4) y_1 and y_2. From (2) will be obtained λ_2, which inserted in (5) will give the pull

in the bars which has to be absorbed by shearing stress in the concrete over an area$=12b$. As it is desirable to take twice the factor of safety in shear that is taken in bending, P_s should not exceed $6bs$, where s is taken at one-fourth the compressive strength of the concrete.

If beams are loaded at two points some distance apart the maximum shearing stress is likely to be of a very different character. The bending moment being uniform between the loading points, the first cracks on the tension flange are as apt to occur under one of the loads as in the middle and this will greatly reduce the strength of the anchorage of the ends of the bars represented by the shearing resistance of the concrete along the plane just above the metal between the crack and the end of the beam. This is especially true as the maximum shearing stress along this plane is likely to be double the average stress. In such cases, as also in cases of uniform load where the shear exceeds the limits above given, the bars should be bent up at the ends as shown in Figs. (1) and (2).

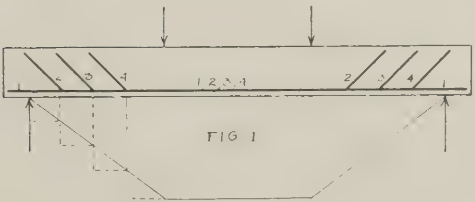


FIG 1

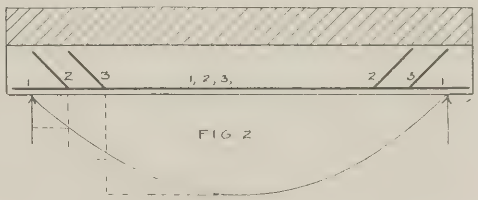
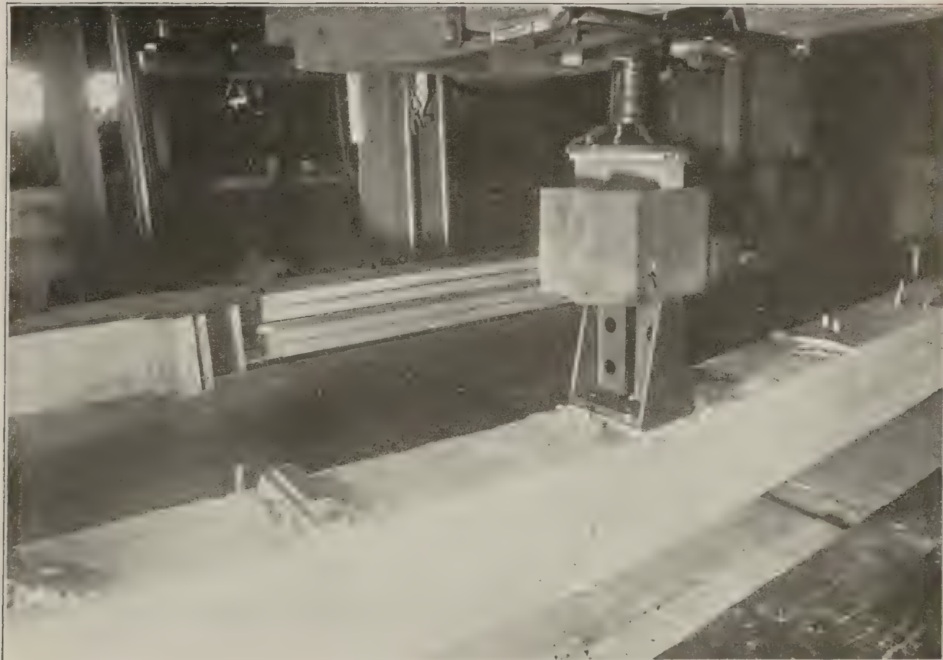


FIG 2

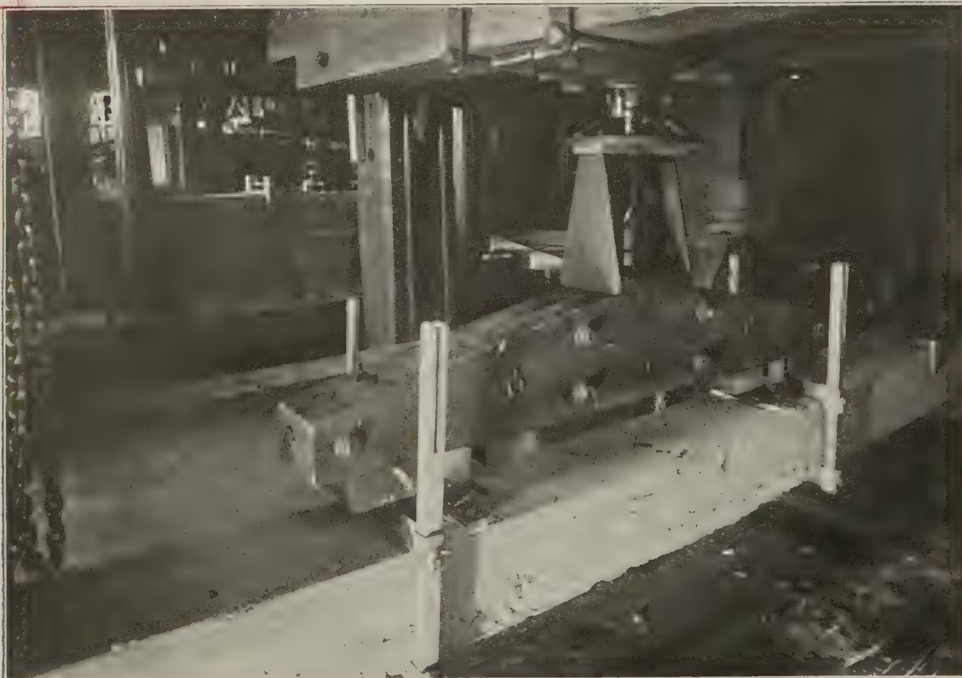
Rock Concrete, 1:2:5; Age 74 days. Depth, 5″; Width, 12″; Span, 10′; Two ½″ corrugated bars=.34□″. Theoretical, Mo=80,600″ pounds; Actual, M=94,200″ pounds. No shear bars used.

Rock Concrete. 1:2:5; Age 72 days. Depth, 7″; Width, 12″; Span, 12′; Three ½″ corrugated bars=.51 □″.
Theoretical, Mo=174,200″ pounds; Actual, M=212,100″ pounds. No shear bars used.

Rock Concrete, 1:2:5; Age 76 days. Depth, 9½″; Width, 12″; Span, 15′; Four ½″ corrugated bars=.68 ″. Theoretical, Mo=322,200″ pounds; Actual, M=402,700″ pounds. Four vertical rods inserted near each end.

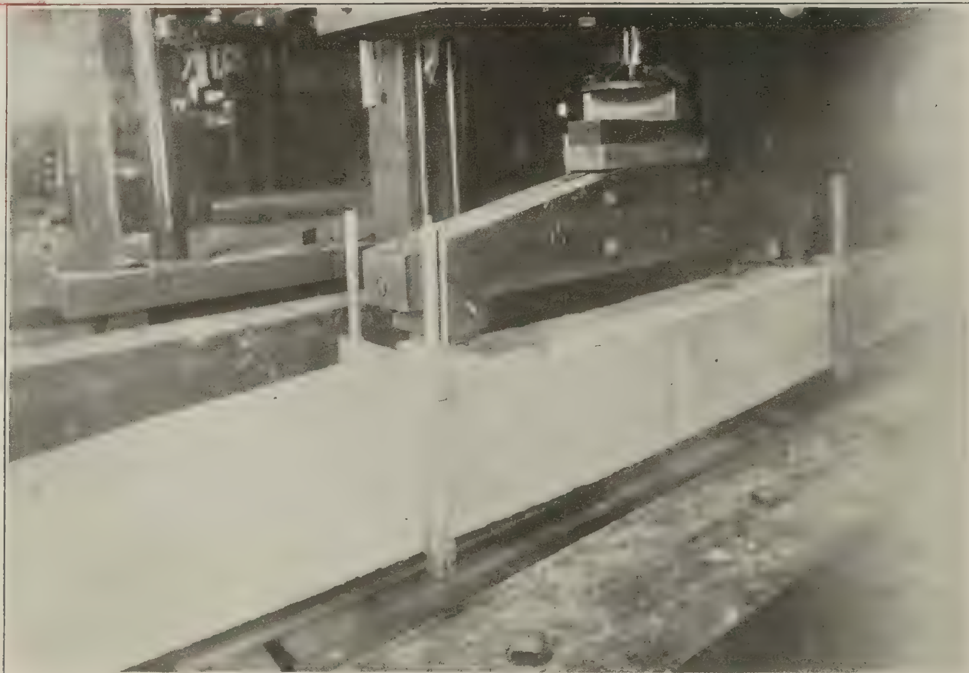


Rock Concrete, 1:2:5; Age 73 days. Depth, 14″; Width, 12″; Span, 15′; Six ½″ corrugated bars=1.02 □″. Theoretical, Mo=725,000″ pounds; Actual, M=929,700″ pounds. Each of the three pairs of horizontal rods bent up vertically at different subdivisions of span.

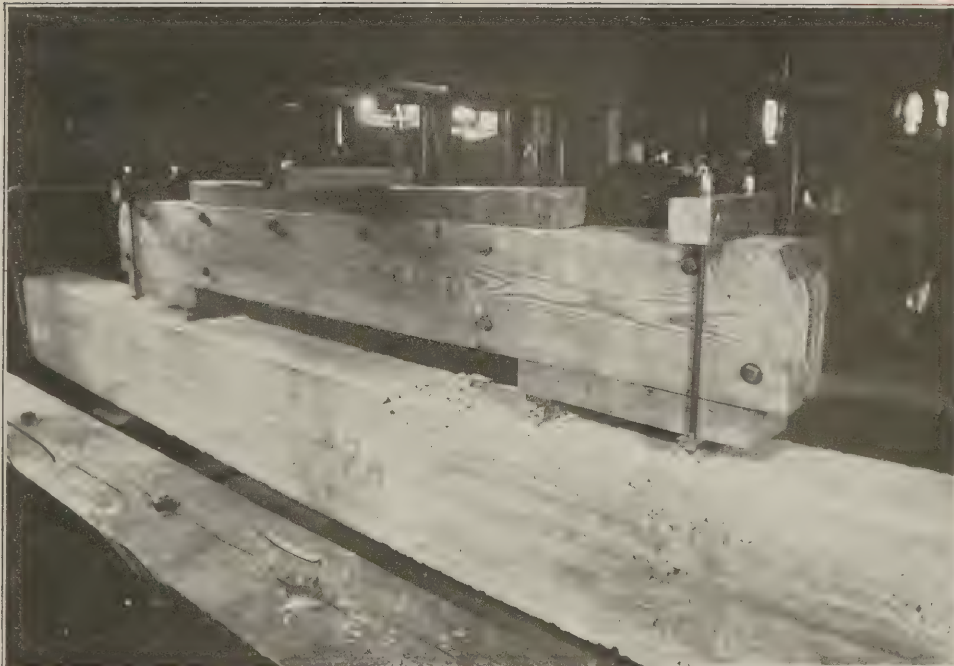


Rock Concrete, 1:2:5; Age 71 days. Depth, 10″; Width, 12″; Span, 12′; Two ¾″ corrugated bars=.62□″. Theoretical, Mo=283,000″ pounds; Actual, M=314,200″ pounds. No shear bars used.

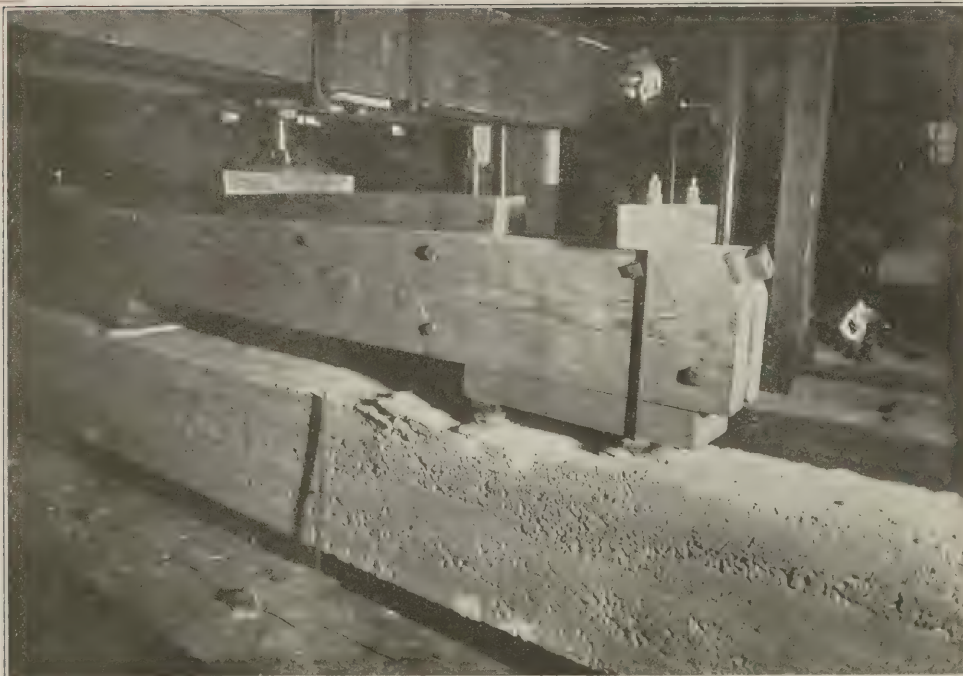
Rock Concrete, 1:2:5; Age 69 days. Depth, 14½″; Width, 12″; Span, 15′; Three ¾″ corrugated bars=.93□″. Theoretical, Mo=625,000″ pounds; Actual, M=637,600″ pounds. Two bars bent up at quarter point which was too close to center for method of testing.



Rock Concrete, 1:2:5; Age 115 days. Depth, 14½″; Width, 12″; Span, 15′; Three ¾″ corrugated bars=.93 ″. Theoretical, M_o=625,000″ pounds; Actual, M=655,000″ pounds. Four vertical bars at each end.



Rock Concrete, 1:2:5; Age 78 days. Depth, 19″; Width, 12″; Span, 18′; Four ¾″ corrugated bars=1.24 ″. Theoretical, Mo=1,121,000″ pounds; Actual, M=1,190,900″ pounds. No shearing provision whatever.



Rock Concrete, 1:2:5; Age 78 days. Depth, 19″; Width 12″; Span 18′; Four $\frac{3}{4}$″ corrugated bars=1.24 □″. Theoretical, Mo=1,121,000″ pounds; Actual, M=1,151,800″ pounds. Four vertical bars at each end.

Rock Concrete, 1:2:5; Age 70 days. Depth, 19″; Width, 12″; Span, 17′ 8″; Four $^3/_4$″ corrugated bars=1.24 ″. Theoretical, Mo=1,121,000″ pounds; Actual, M=1,142,300″ pounds. Four vertical bars at each end.

Rock Concrete, 1:2:5; Age 77 days. Depth, 14″; Width, 12″; Span, 15′; Two ⅞″ corrugated bars=1.1[illegible]″. Theoretical, Mo=725,000″ pounds; Actual, M=756,500″ pounds. Four vertical bars at each end.

Rock Concrete, 1:2:5; Age 75 days. Depth, 18″; Width, 12″; Span, 18′; Two 1″ corrugated bars=1.4 .″. Theoretical, Mo=1,177,800″ pounds; Actual, M=1,149,300″ pounds. Four vertical bars at each end.

Test Made at the Brooklyn Navy Yard. Failed by Crushing on Top.

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